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CIRIA RESEARCH PROJECT 407

DESIGN OF STEPPED BLOCK SPILLWAYS

Final Draft

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DESIGN OF STEPPED-BLOCK SPILLWAYS

CONTENTS

NOTATION

a	Height of lip of deflector block above invert of tailwater channel
B	Width of block transverse to flow
C	Discharge coefficient of weir
Cu	Undrained shear strength
C^1	Apparent cohesion (in terms of effective stress)
D_{15}	Particle size not exceeded by 15% by weight of particles in upper layer
d_{85}	Particle size not exceeded by 85% by weight of particles in lower layer
Fr	Froude number
Fs	Factor of safety
F_o	Frictional force resisting sliding of block
F_1, F_2	Frictional forces resisting vertical movement of block
g	Acceleration due to gravity
H	Upstream head over weir
i	Hydraulic gradient
J	Longitudinal slope of channel ($= \tan \theta$)
k	Permeability
L	Exposed length of top surface of block when assembled
L_o	Overall length of block
M	Mass of block
O	Opening size in geotextile
P_g	Force on end of block due to pressure in gap
P_1	Pressure force on lower surface of block
P_s	Pressure force on step
P_u	Pressure force on upper surface of block
P	Vertical distance of water surface below crest of spillway
q	Discharge per unit width

q_f	Value of q at failure
R_o	Normal reaction from underlayer
R_1, R_2	Normal reactions from adjacent blocks
s	Longitudinal slope of channel (= sin θ)
T_g, T_h	Shear forces due to flow in gaps between blocks
T_l	Shear force on lower surface of block due to seepage flow
T_u	Shear force due to flow over upper surface of block
t_a	Mean thickness of block
t_n	Thickness of block measured normal to slope of channel
t_s	Thickness of block measured normal to top surface of block
U	Coefficient of soil uniformity
V	Mean velocity of flow
y	Depth of flow measured normal to slope of channel
y_c	Critical depth
y_t	Depth of flow at tailwater
Z_o	Vertical distance between upstream and downstream water levels
Z_t	Vertical distance between upstream water level and toe of spillway
Δ	Height of step
Δ_n	Value of Δ measured normal to slope of channel
Δ_s	Value of Δ measured normal to top surface of block
η	Permeability reduction factor
θ	Angle of channel to horizontal
Φ	Flow resistance parameter
ϕ'	Angle of shearing resistance (in terms of effective stress)
ψ	Permittivity

1. INTRODUCTION

1.1 Background

The original concept of utilising stepped elements for the protection of erodible surfaces subject to high velocity flow arose from work carried out by Dr. P.I. Gordienko at the Moscow Institute of Civil Engineering in the late 1960's. When investigating the possible use of wooden boards as a protection medium, he observed that an overlapping (so-called 'clinker built') form of construction was more stable than planks simply butt-jointed to one another.

The development of this concept into a workable engineering solution is due principally to the work of Professor Yuri Pravdivets, also at the Moscow Institute of Civil Engineering. He took up the concepts of Gordienko in the early 1970s and has undertaken a continuing programme of laboratory tests, theoretical studies and prototype trials which has resulted in the successful construction of eight fully-operational stepped-block service spillways on embankment dams in the USSR. Details of these spillways, some of which have now been in operation for more than 10 years, are given in Appendix A1.

Various other Russian researchers asserted Pravdivets in his studies, but he led the practical development of the concept in the USSR. He held patents on the blocks but these have now devolved to the State, and are only applicable inside the USSR. There are therefore no restrictions on their use outside the USSR.

Outside the USSR, relatively little attention had been given by practitioners to wedge-shaped blocks prior to the CIRIA appraisal study. Research has been carried out by Jiang (Tong Ji University, Shanghai, China), El Khashab (King Faisal University, Saudia Arabia) and Noori (University of Southampton, UK) and the Chinese work resulted in the construction of a stepped-block spillway. The lack of first-hand understanding and experience clearly has however been a constraint on their use in the West where there is a natural reluctance to use unfamiliar techniques.

The CIRIA study arose out of the following:-

(a) interests of engineers in the UK and US in low-cost methods of upgrading spillway or embankment overtopping capacity on existing dams where spillway capacity was known to be inadequate in the light of current hydrological predictions. The number of existing dams in Great Britain being modified to provide additional spillway capacity has increased since the implementation in 1985 of the Reservoirs Act 1975.

(b) experience in the UK and US of the limitations in the performance of plain, parallel sided concrete blocks and other low-cost protection systems to spillways with high velocity flow.

Particular interest existed in developing a flexible but relatively robust protection system suitable for high intensity discharge on service spillways or diversions subject to long-duration flow. Protection systems presently used in the West for high velocity applications include roller compacted concrete, rip-rap, gabion mattresses, cable-tied concrete blocks and soil/cement stabilisation. The study is seen as complementary to the CIRIA study on reinforced grass which was developed under the UK Reservoir Safety research programme for low intensity discharge (up to about 1.5 $m^3/s/m$) on auxiliary spillways or dam embankments subject to overtopping, the results of which were published in CIRIA Report 116 (1987).

Since the CIRIA study commenced, laboratory tests have been undertaken in the UK at Salford University, both as part of the CIRIA study and also for a British consultant designing a stepped-block spillway to be constructed in Oman. In the US, tests on stepped blocks were carried out in a large scale flume for the Federal Highway Administration and the US Bureau of Reclamation by Simons, Li and Associates Inc. Professor Pravdivets visited the UK in 1989 and 1991 and the US in 1990 to advise the personnel undertaking these tests.

1.2 Scope

This design guide provides a framework within which an experienced dam engineer can consider the use of stepped blocks as a form of spillway construction or protection to embankments subject to overtopping. CIRIA emphasises the strategic nature of spillways and embankment protection, and warns against the uninformed use of apparent 'low-cost' protection - such as stepped blocks - without the comprehensive consideration which this guide recommends. In this respect, it should be noted that many spillway failures occur not through direct failure of the primary protection to withstand the flow, but through problems - often due to poor attention to detail - at the edges of the protection, or in the underlayer or subsoil.

This report is based on the best information available at the time of writing. In several areas the scientific understanding of the processes involved is still incomplete and yet to be improved by future research and experience. In such areas the report has sought to indicate the present limitations, and to set out an approach which is appropriately conservative as well as being generally acceptable to the practising engineer.

2. BASIC FEATURES, APPLICATIONS AND ADVANTAGES OF STEPPED-BLOCK SPILLWAYS

This section summarises the constituent elements of a stepped-block spillway and the functions which they fulfil. Possible modes of failure are discussed and examples of applications of stepped-block spillways are given.

2.1 Basic Features

2.1.1 Principal elements

The general features of a stepped-block spillway are shown in Figure 2.1. The principal elements are as follows:

1. Stepped blocks. These are made of precast concrete and may be

   (a) overlapping and wedge-shaped
   (b) butt-jointed and wedge-shaped
   (c) overlapping slabs

   Details of these three types of block are given in Fig. 2.1 and a photograph of an overlapping and wedge-shaped block used for model testing is shown in Fig. 2.2.

   Overlapping joints have the advantage of providing positive interblock restraint. Successive rows of blocks are normally laid in stretcher bond configuration with side joints between successive rows staggered: this avoids flow concentration at the joints. Individual blocks do not need to be positively connected to neighbouring blocks to achieve hydrodynamic stability but this can be done to reduce the risk of theft or vandalism. Blocks must contain holes or grooves to permit drainage of the underlayer and reduce excess pore water pressures in the subsoil. All the blocks used in the Russian installations were reinforced. Further features of the different types of block are given in Section 6.2.

   The size of the block and the type of joint depend on the following considerations:

   - hydrodynamic forces;
   - flexibility to accommodate limited differential settlement of subsoil and/or underlayer. A positive compressive stress must always be maintained on the subsoil/underlayer;
   - method of construction and provision for repair and replacement;
   - durability. Spillways can be subjected to freeze/thaw conditions. Unless positive measures are taken to restrict construction or maintenance vehicles from the chute, live vehicular loading should be considered;

- possible disruption by vegetation growing up through the joints;

- threat of vandalism or removal of blocks.

2. Underlayer. This may comprise two or more layers of granular material, or a combination of granular material and a geotextile. In small installations on low-permeability subsoils where spillway operation is infrequent, a geotextile alone may be sufficient.

   The functions of the underlayer are as follows:

   - to assist the relief through drainage vents of excess hydraulic pressure at any location below the block;

   - to restrain soil particles on the subsoil formation against movement due to seepage exiting from the subsoil;

   - to provide an even foundation for the stepped blocks.

   The underlayer must also protect the subsoil from erosion by drainage flow in the underlayer parallel to the slope. This function is generally satisfied by the filter requirements.

3. Subsoil. This may comprise either original ground or fill material. Seepage flow will cause partial or complete saturation and could affect subsoil stability. The seepage flow will depend on the permeability of the subsoil and the existence of any cut-off or core preventing seepage from upstream of the spillway. Provision for seepage flow may be either nominal in the case of low-permeability clayey soils or must be purpose designed for higher permeability granular soils such as gravel. Seepage flow and slope stability need to be considered during construction and before, during and after flow occurring down the spillway.

2.1.2 Forces on blocks

The principal forces acting on an individual wedge-shaped and butt-jointed block in a stepped-block spillway are shown diagrammatically in Figure 2.3 and comprise the following:

- normal pressure force on upper surface of block (P_u);

- shear force on upper surface (T_u);

- normal pressure force on step (P_s);

- normal forces on ends of block due to pressure in gap (P_g);

- shear forces on blocks due to flows in gaps (T_g, T_h);

- uplift pressure force on lower surface of block (P_1);

- normal reaction forces from adjacent blocks (R_1);
- frictional forces resisting vertical movement of block (F_1, F_2);
- shear force due to seepage flow beneath block (T_1);
- normal reaction from underlayer (R_o);
- frictional force resisting sliding of block down embankment (F_o);
- the weight of the block (Mg where M is the block mass and g is acceleration due to gravity).

Each of the flow-induced forces can be considered to consist of a steady mean component and a fluctuating component. The block will move if the disturbing forces or moments exceed the corresponding restraining forces or moments.

2.1.3 Modes of failure

The laboratory studies have shown stepped-blocks to be extremely stable under high discharges and little information has therefore been obtained about ways in which such systems could fail.

A 'failure' should be deemed to have occurred if it causes, or threatens to cause, significant damage to the structure or material which the blocks are intended to protect. Failure would therefore include the removal of blocks or large deformations that expose the underlying material to serious erosion. On this basis, minor adjustments in the gaps between blocks, or small-amplitude vibrations due to turbulence in the flow should not be considered as 'failures' if they do not result in any damage to the underlayer. A suitable definition for failure of a block is therefore a movement or deformation large enough to produce a sustained loss of contact between the block and the underlying formation.

The following potential modes of failure, illustrated in Fig. 2.4 have been identified:

(a) Sliding of blocks. If the blocks are initially assembled with gaps between them in the longitudinal direction, then a block might start to slide down the slope. The limit of stability will be reached when the resultant of the forces acting on a block parallel to the slope exceeds the frictional resistance between the block and the top of the underlayer. In practice, a block will only slide until it bears up against the adjacent block downstream, so if all the blocks slide in this way it would expose the underlayer at the top of the spillway and increase the load on the toe.

This type of failure can be prevented by either ensuring that blocks are laid without gaps between them or providing shear keys to anchor the blocks to the subsoil at intermittent points and a toe block at the downstream end of the stepped-blocks.

(b) Lifting of blocks. If the net pressure and shear forces acting on a block normal to the slope of the channel exceed the corresponding component of its own weight, then lifting of the block may cause failure to occur. If the blocks are close together or they have overlapping joints, this mode of translational failure may be unlikely except in a hydraulic jump.

(c) Rotation of blocks. Even if the normal and tangential forces acting on a block are in equilibrium, they may produce an out-of-balance moment which causes it to rotate so that one end is lifted into the flow. In the case of a wedge block, the pressure at the upstream end (in the separation zone) is lower than at the downstream end so it will tend to rotate about its downstream edge. In a hydraulic jump, the rotation may be about the upstream edge of the block.

(d) Deep-slip failure. This is a failure within the subsoil caused by penetration of water from the spillway leading to a reduction in shear strength. The conditions for failure can be analysed by standard slope-stability methods.

(e) Shallow-slip failure. This is a slip along a plane parallel to the face of the embankment caused by the down-slope forces on the blocks and an adjacent layer of soil exceeding the local shear resistance along the underside of the soil layer.

(f) Settlement failure. Incorrect design of the underlayer and/or drainage holes could allow flow to remove material from the underlayer and thereby result in settlement and block displacement. Settlement of an embankment could cause gradual displacement of the blocks so that they no longer function correctly when subjected to flow.

When considering the failure of a block due to movement (modes (a) (b) and (c) above), two possibilities can be envisaged. Firstly, the steady mean forces exerted on a block by the flow may exceed the resisting forces so that movement occurs as soon as the discharge reaches a certain intensity. Secondly, the block may be stable under the action of the mean forces but may move when there is an unfavourable combination of fluctuating forces. Thus, for example, a block may slide if a sudden increase in shear stress on its upper surface coincides with a sudden reduction in the normal pressure (reducing the normal reaction and frictional resistance from the underlayer). Movement will, therefore, occur randomly and its probability will depend upon how extreme a combination of turbulent fluctuations is required.

Movement having occurred, it is then necessary to consider whether the block is in a state of stable or unstable equilibrium. If a

sudden turbulent fluctuation causes the block to lift or rotate slightly, its position relative to the other blocks will be altered. As a result, the change in the flow pattern around the block will affect the mean and turbulent forces acting on it. If these forces tend to become smaller, the block is in stable equilibrium and will not fail in this mode. However, if the initial movement causes the disturbing forces to increase, displacement of the block will continue and failure will occur.

Further information relating to geotechnical failures (modes (d) (e) and (f) above) can be found in Section 5.

2.2 Applications

Much of the Russian research on stepped-blocks was directed towards high prototype velocities up to 100 $m^3/s/m$, which is equivalent to an overtopping head of about 15m. It is likely that typical applications in the U.K. would be much smaller with overtopping heads of 2 or 3m and flows up to about 10 $m^3/s/m$.

Stepped block protection can be used in a number of situations associated with dams, reservoirs and flood control. Examples of typical applications are:

(a) for dam chute spillways. These can be either main spillways or auxiliary ones which only discharge in extreme flood events. They may be constructed either in existing ground through the dam abutment or on the dam embankment itself where its flexibility and low weight should overcome any post-construction settlement;

(b) on the downstream side of embankments which could be subject to overtopping during extreme flood events such as flood storage embankments, flood embankments along low-lying rivers and road embankments;

(c) temporary spillways to convey floods of magnitude greater than the design diversion discharge across the downstream face of a partly completed dam embankment. In the case of a new embankment dam having a construction programme longer than a year and a seasonal close-down of construction during the flood season, economies can be made if the partly constructed spillway can be used as a temporary spillway during the close-down period. A stepped-block spillway is particularly advantageous in such a situation since it can be constructed at the same rate as the embankment and can be temporarily capped off.

Other possible applications are on road embankments occasionally subjected to overtopping, barrages and irrigation structures.

Examples of typical applications are illustrated in Figure 2.5 and some descriptions and photographs of prototype installations are given in Appendix A1.

2.3 Advantages of stepped-block spillways

Stepped-block protection has the following basic advantages:

- the upstream edge of the block is shielded from experiencing potentially-disruptive flow stagnation pressure, which can otherwise give rise to extreme lift and drag forces on a protection system;

- the flow pattern causes a low-pressure separation zone downstream of each step (see Fig. 2.1). This zone is connected by drainage vents to the underlayer and controls the build-up of seepage flow;

- the block shape is inherently stable. If any block tends to move off the slope (either perpendicular to the slope, or by rotation about one end), the sloping upper surface experiences a stabilising downthrust;

- the stepped upper surface has a high roughness which helps to dissipate the energy of the flow and reduce flow velocity, hence reducing the amount of energy to be dissipated at the tailwater or toe.

2.4 Economics

Russian experience has shown that the cost of stepped-block spillways is potentially competitive in comparison with other forms of construction such as in-situ reinforced concrete, flat concrete slabs, rip-rap or roller-compacted concrete. The main economic advantages of stepped-blocks are as follows:

(a) the on-site works can be reduced to relatively straightforward and low-skill activities;

(b) a reduction in the time taken to construct the spillway and associated works. The scheme shown in Fig. 2.4b for the temporary diversion of floods across a partly constructed dam is based on a Russian feasibility study which concluded that use of the temporary stepped-block spillway could reduce the overall construction period by a year because of the reduction in the diversion works required beneath the dam;

(c) the installed cost of stepped blocks per unit discharge is substantially lower than for other materials where a much greater thickness is required. For example a typical stepped-block spillway with average thickness of 0.2m would pass a flow of 10 $m^3/s/m$ while a conventional in-situ concrete spillway chute would normally be at least 0.5m thick to pass this flow. There would also be a considerable reduction in cost of the toe, where precast blocks could be used instead of a large amount of in-situ concrete.

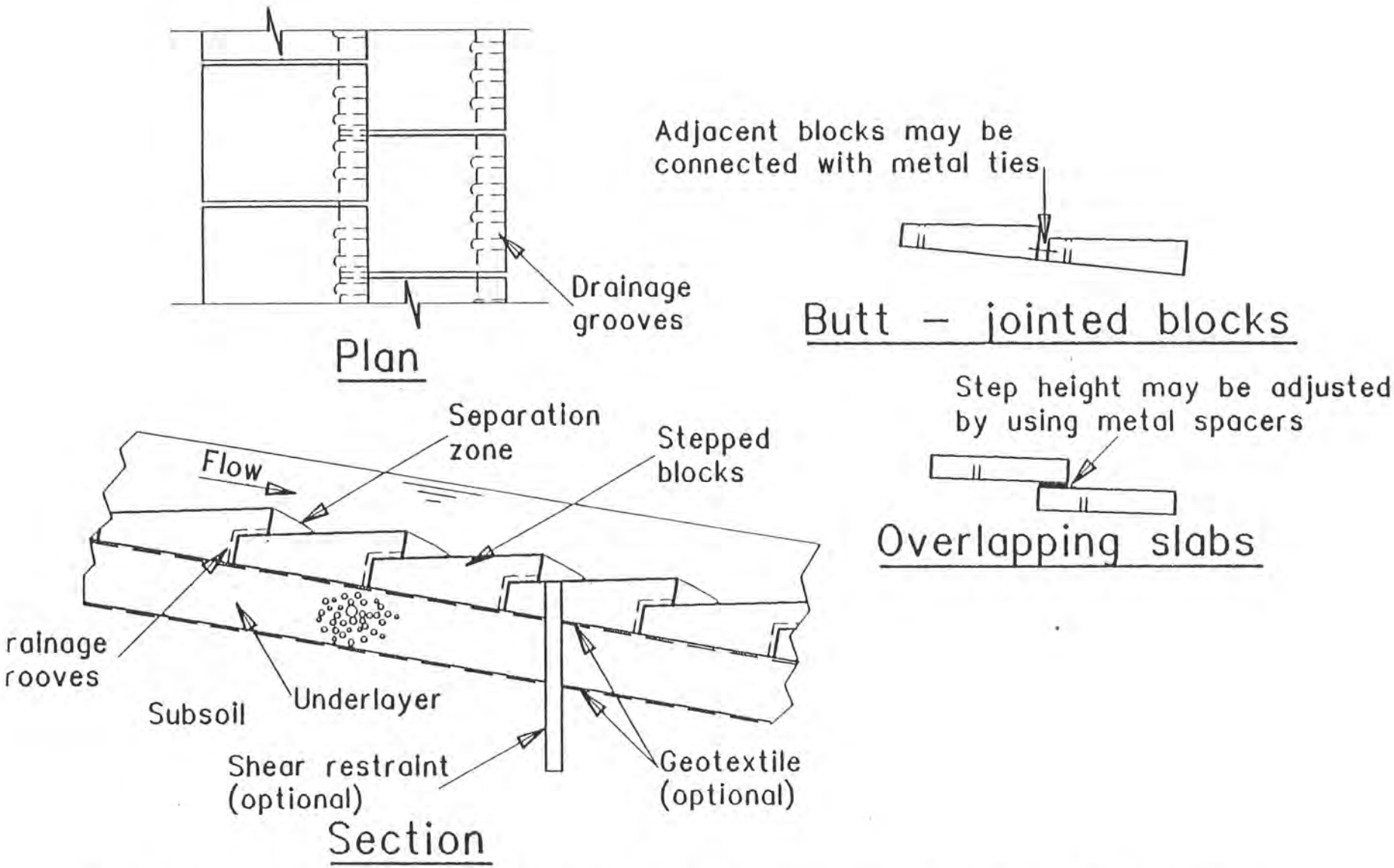

Fig 2.1 General Features Of Stepped Block Spillways

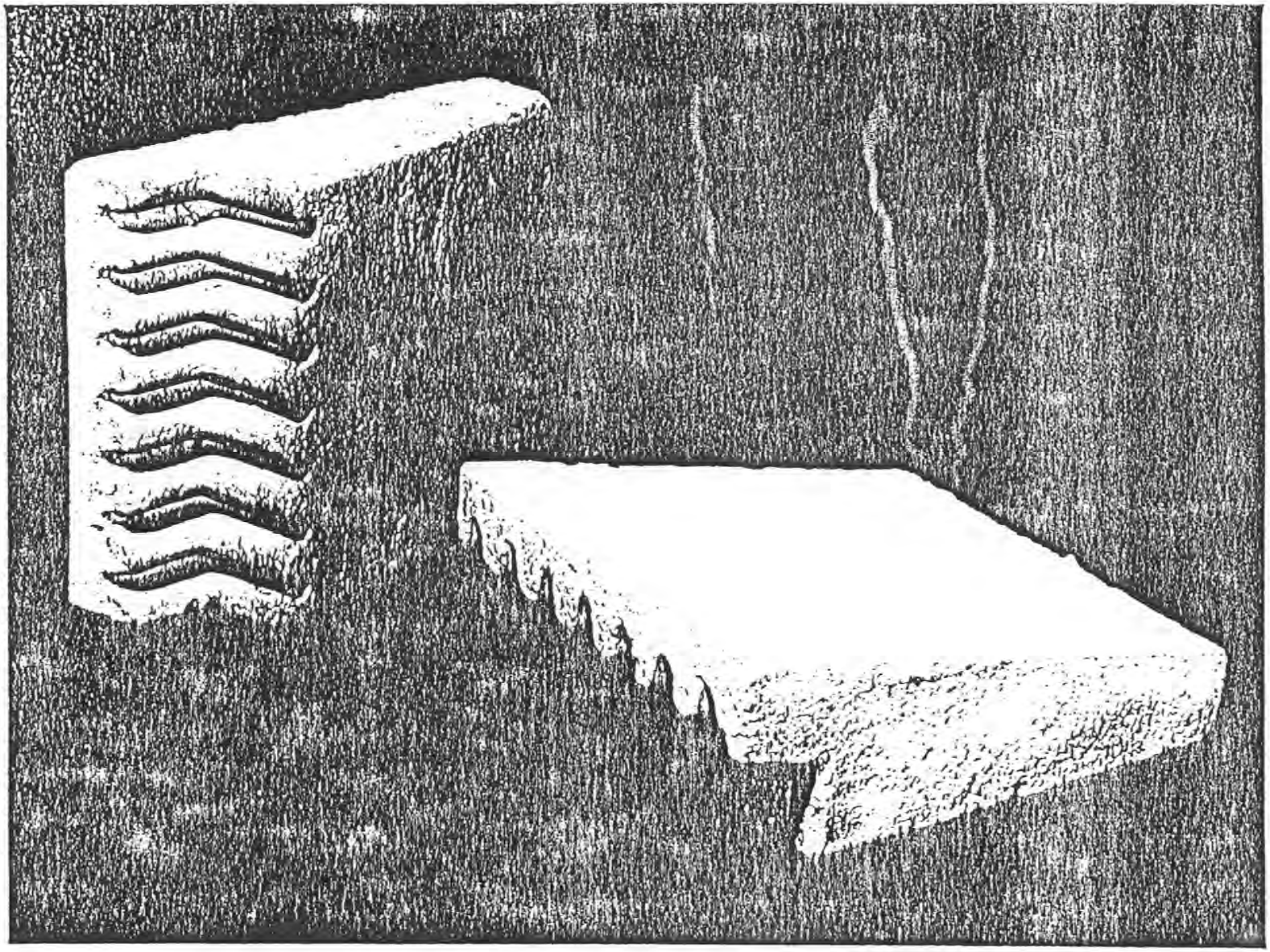

Fig 2.2: Blocks used for model testing.

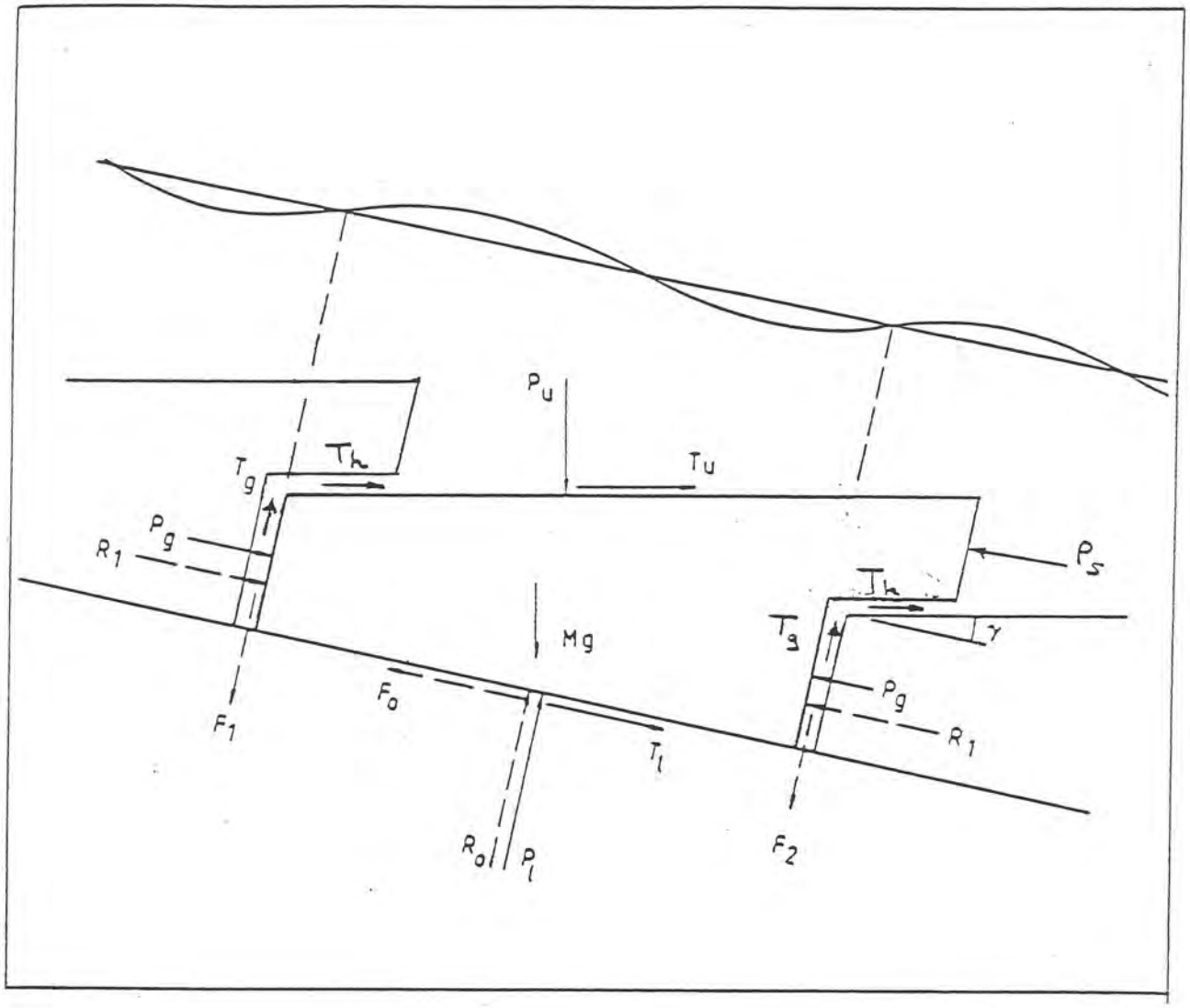

Fig 2.3 : Forces on blocks

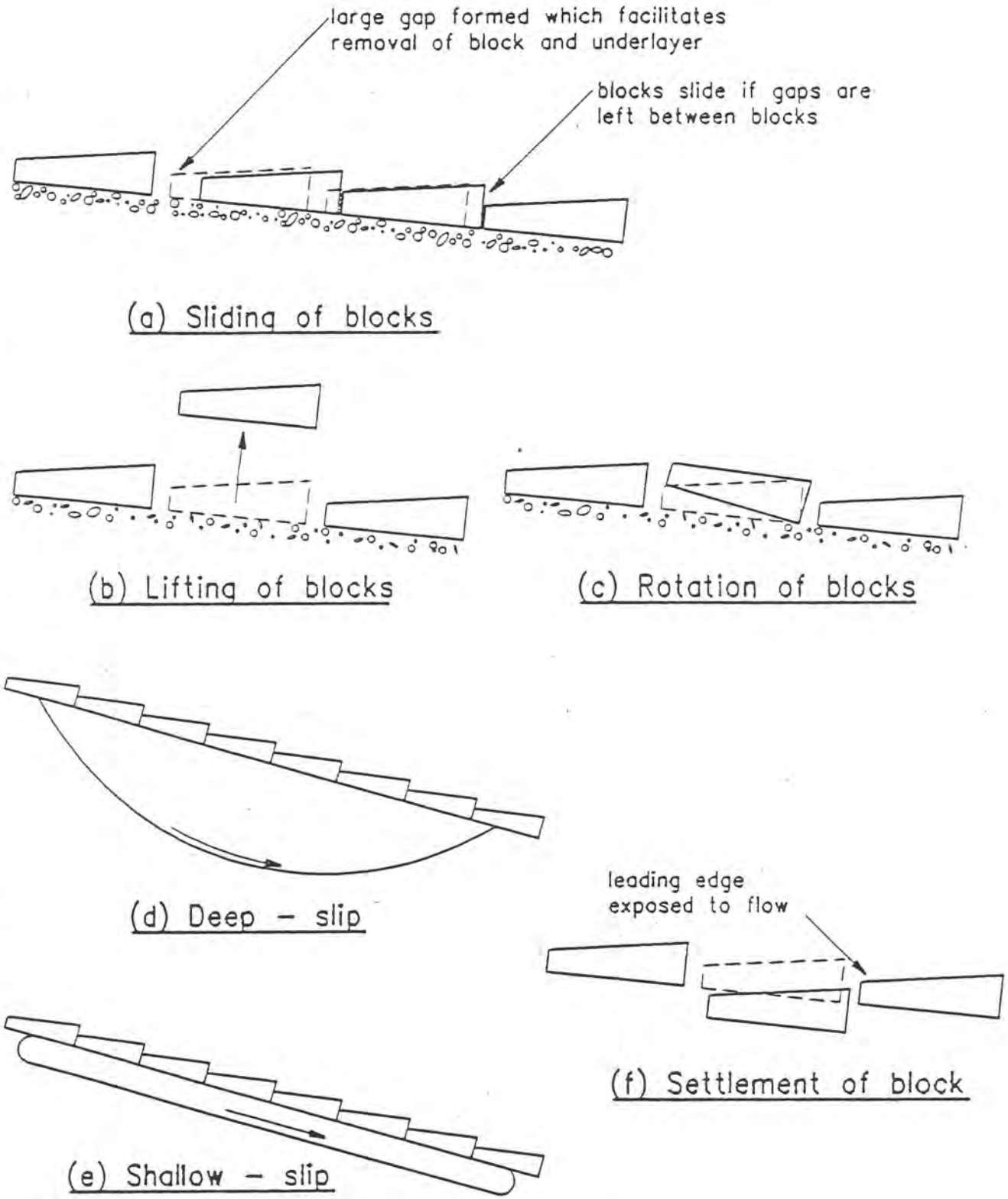

Fig2.4 Modes Of Failure

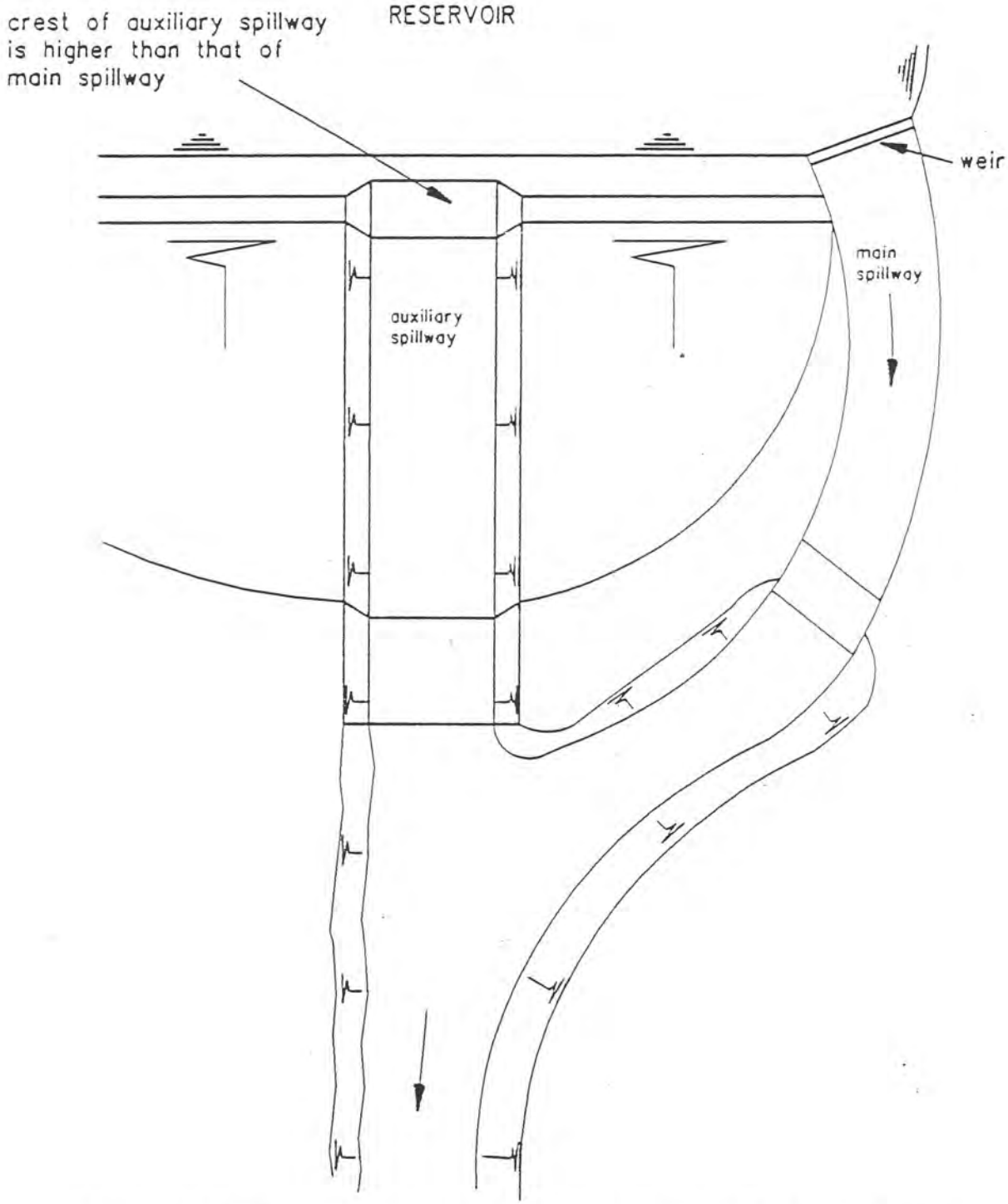

Fig 2.5a Auxiliary Spillway For Dam

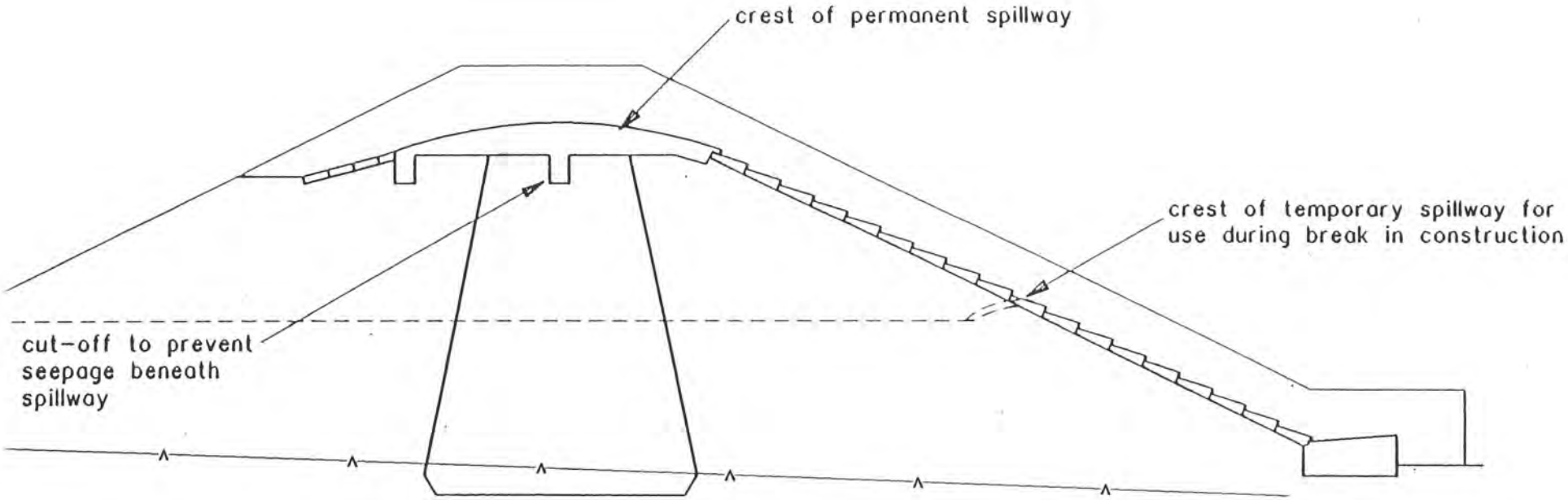

Fig 2.5b Permanent Spillway Adapted For Temporary Use During Construction

3. PLANNING AND DESIGN PROCEDURE

The design of a stepped-block spillway cannot be considered in a wholly analytical manner. It cannot be fully standardised; every spillway requires competent engineering design input and judgement which addresses the specific problems of the particular site. This section provides a summary of the points that need to be considered in the planning and design process.

3.1 Planning

The planning stage involves consideration of the basic design parameters and determining the most appropriate type of spillway at a particular site. Among the alternatives considered could be reinforced concrete, reinforced grass, roller compacted concrete, gabions, flat concrete slabs or stepped blocks.

A summary of the advantages and disadvantages of stepped blocks is given in Table 3.1.

Table 3.1 Features of stepped block spillways

Advantages	Disadvantages
Relatively simple and quick construction.	Little prototype experience in the West.
Hydrodynamically stable.	More design consideration required than for other types of spillway.
High unit discharge.	
No limit to duration of flow.	
Flexibility.	
Low maintenance.	

The following points should be considered at the planning stage:

- Frequency, duration and quantity of flow
- Risk (acceptability of failure)
- Properties of subsoil
- Usage of area around spillway (e.g. agricultural or amenity use, risk of vandalism)
- Capital and maintenance costs
- Appearance
- Maintenance ability and requirements of owner
- Access across spillway
- Access to site and method of construction
- Climate

3.2 Design procedure

The detailed design of a stepped-block spillway needs to be considered in terms of (a) the overall spillway and the stability of its foundations and (b) the individual elements of the spillway, in particular the channel cross-section, the block, crest and toe details and the underlayer design.

In addition to producing construction drawings, it will also be necessary at the design stage to prepare a specification and plan for future inspection and maintenance.

The main items to be considered in the detailed design process are as follows:

- Estimation of design outflow. This will include choosing the return period of the maximum design inflow hydrograph: in the case of dam spillways in the UK, categories A, B, C and D in the ICE Engineering Guide would be used to determine the design flood. Different spillway widths and weir crest levels may be considered, with a flood routing calculation being undertaken for each option to determine the maximum upstream water level and flow down the spillway.

- Determination of the tailwater level/discharge relationship for a range of flows from low flows to maximum design discharge: this information will be needed when designing the toe details.

- Preparation of a preliminary general arrangement of the spillway, with the preferred spillway width and weir crest level. (Section 6.1).

- Undertaking a ground investigation to obtain detailed information on the nature of the subsoil. This will then be used to determine the seepage conditions in the subsoil before, during and after flow down the spillway. Stability analyses to determine the safe slope on which the spillway can be constructed will then be undertaken using this information. (Section 5.2).

- Choosing the shape and size of stepped block to be used. This will depend on a number of factors including discharge capacity, threat of vandalism, ease of construction and cost. (Section 6.3).

- Determination of hydraulic roughness. This is needed in order to calculate the maximum depth of flow and check that the spillway sides are high enough to contain the flow, and also for design of the toe details. (Section 6.4).

- Detailed design of:

  - spillway sides (Section 6.6)
  - spillway crest, including access requirements (Section 6.8)
  - energy dissipation at tailwater/toe and downstream of the spillway (Section 6.9)
  - underlayer (Section 6.7)

- Preparation of a specification for supply of the materials and construction of the spillway. (Section 7).

- Preparation of a plan for inspection and maintenance of the spillway once it has been constructed. (Section 8).

The design procedure is illustrated in Figure 3.1.

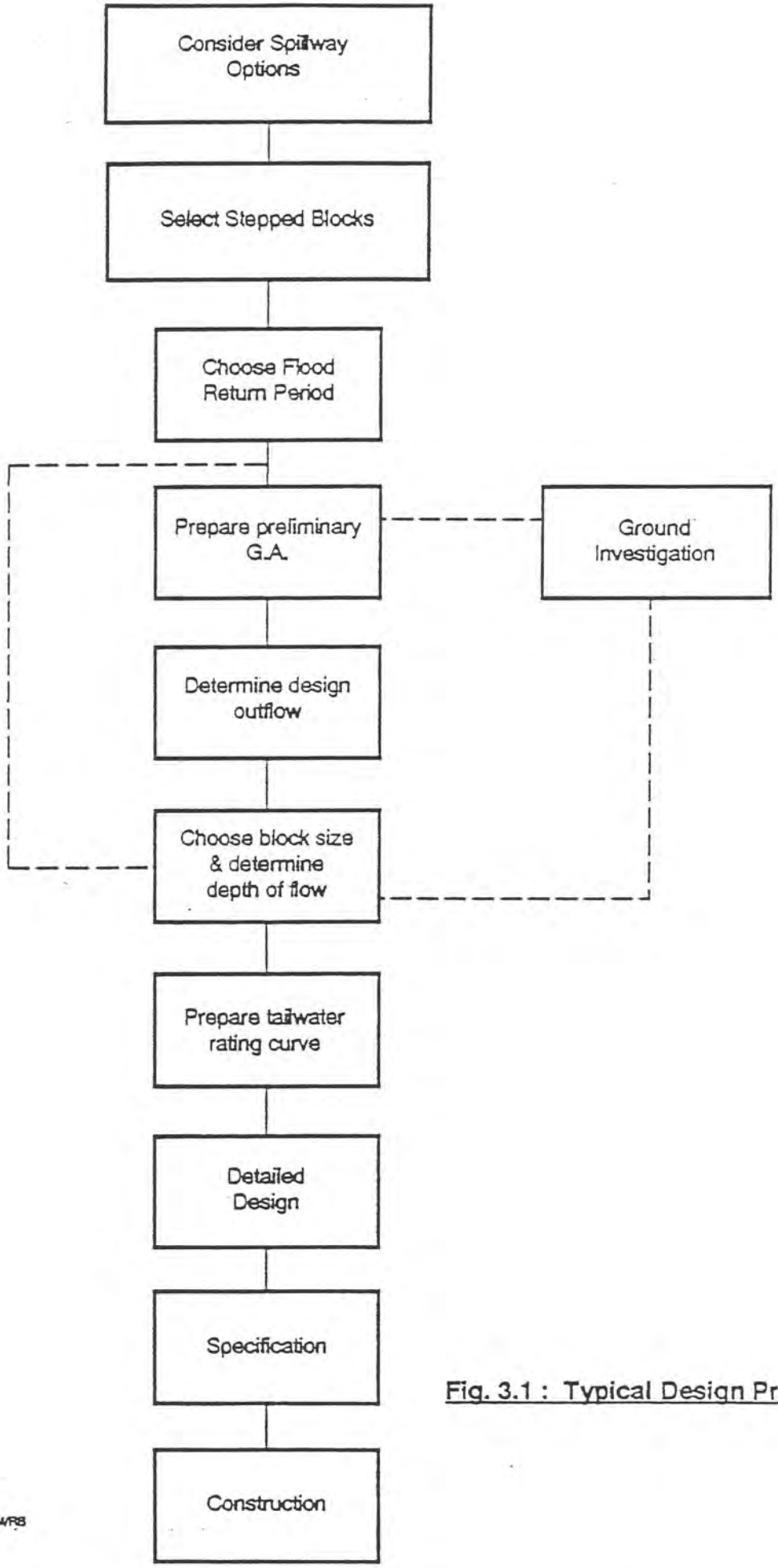

Fig. 3.1 : Typical Design Procedure

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4. HYDRAULIC CONSIDERATIONS

4.1 Introduction

This section has been included to give the reader a basic understanding of the hydraulics of open channel spillway flow. More detailed information can be obtained from standard hydraulics reference books such as Chow, Henderson or King and Brater. Specific details of flow down stepped spillways can be found in CIRIA Report 33 (1978).

The general features of flow down a typical stepped-block spillway are shown in Figure 4.1. Uniform flow conditions are achieved when the energy loss due to friction is matched by the rate of loss of potential energy. The spillway need not maintain a constant slope throughout its length: for stepped block spillways it can be beneficial in terms of the flow regime at the tailwater for the slope to be flatter at the downstream end.

4.2 Flow over crest

Flow passes through critical depth control at the crest of the waterway and accelerates downstream in high velocity supercritical flow. Control is normally by means of a weir, which can be shaped to operate most efficiently at the maximum head. Flow over the crest may be restricted by piers, the ends of which should be rounded or pointed.

4.3 Flow down spillway

Some distance downstream of the crest, turbulent boundary layer growth and air entrainment will develop throughout the flow (see Figure 4.2). The degree of turbulence at any point can be considered as a measure of the instantaneous deviations in fluid particle velocity from the mean. The turbulent velocity fluctuations occur in all three dimensions and give rise to corresponding fluctuations in pressure within the flow. Turbulent conditions at the water surface will promote air entrainment, which attenuates pressure fluctuations in the flow. Pressure fluctuations are therefore likely to be greatest in the upper chute region where (a) terminal velocity has been achieved but (b) air entrainment has not penetrated to the bed.

In the supercritical flow region, the velocity head can be substantial (e.g. 5m for flow with mean velocity 10 m/s). It is possible for any local variation in roughness (such as the protruding face of a block in a damaged area of the chute) to experience considerably higher than average lift and drag forces by mobilising locally the velocity head of the flow. The stepped block shape avoids this, however the designer and operator of the spillway should always be aware of these potential effects in order to ensure they are avoided (e.g. by careful detailing and by diligent management).

4.4 Hydraulics at toe of spillway

Flow conditions change abruptly at the tailwater where unidirectional supercritical 'free flow' impinges on the downstream flow regime. There must be a sufficient depth of water for a hydraulic jump to form so that the flow regime reverts to subcritical flow, thus minimising the energy of flow in the channel downstream.

Possible flow conditions at the toe where no energy dissipating device is provided are shown in Figure 4.3. It is probable that more than one of these conditions will be encountered over the full range of possible discharges: where there is a substantial variation in tailwater level with flow, the position of the hydraulic jump will vary accordingly.

If the tailwater depth is insufficient to cause a hydraulic jump to form at the toe, then supercritical flow will continue downstream until sufficient energy has been lost to enable a hydraulic jump to form. The high velocity of flow is likely to cause significant erosion downstream if the channel is unprotected.

A high tailwater level will result in a plunging hydraulic jump forming on the slope of the spillway. The high turbulence of the flow in the jump means that flow is no longer unidirectional and the effectiveness of the stepped block shape is substantially reduced with reverse flow producing drag forces on the steps. The fluctuations in pressure will be much greater in the hydraulic jump which could lead to blocks becoming dislodged. The downstream end of the spillway can be designed to ensure that a surface jump forms instead of a plunging one by flattening the spillway slope upstream of the tailwater and constructing a block at the toe to direct the flow away from the channel bed. This is also shown in Figure 4.3.

Research has shown that failure of a stepped-block spillway is most likely to occur in the region of the hydraulic jump and it is preferable for the toe to be designed to ensure either that the blocks are heavy enough to stay in position or that they are not subjected to a plunging hydraulic jump.

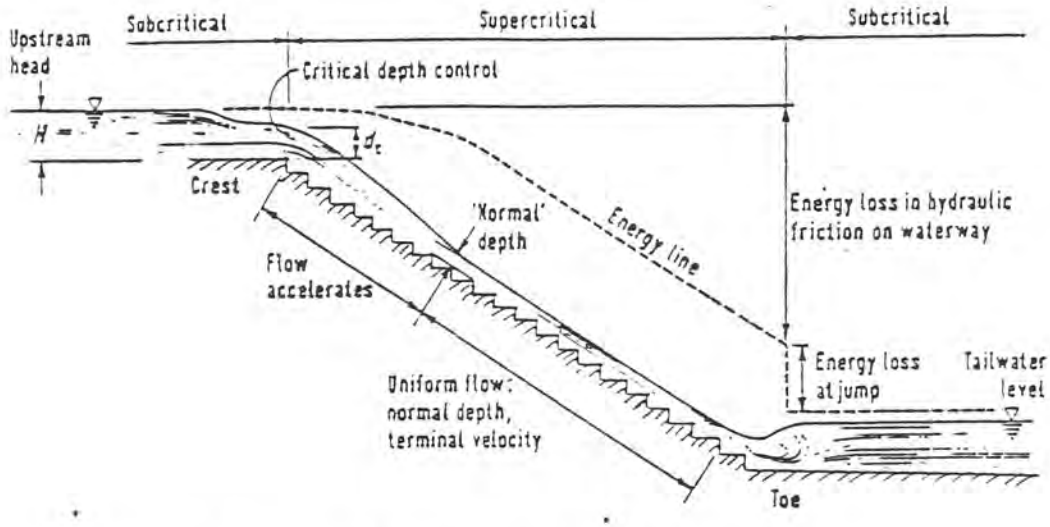


Fig 4.1: General Features of Flow

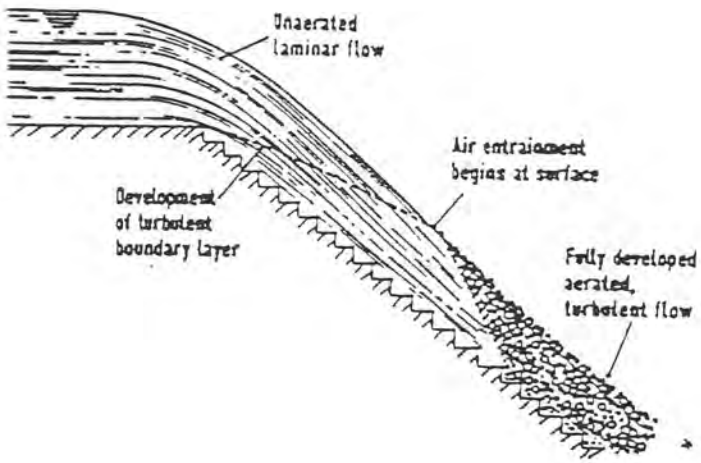
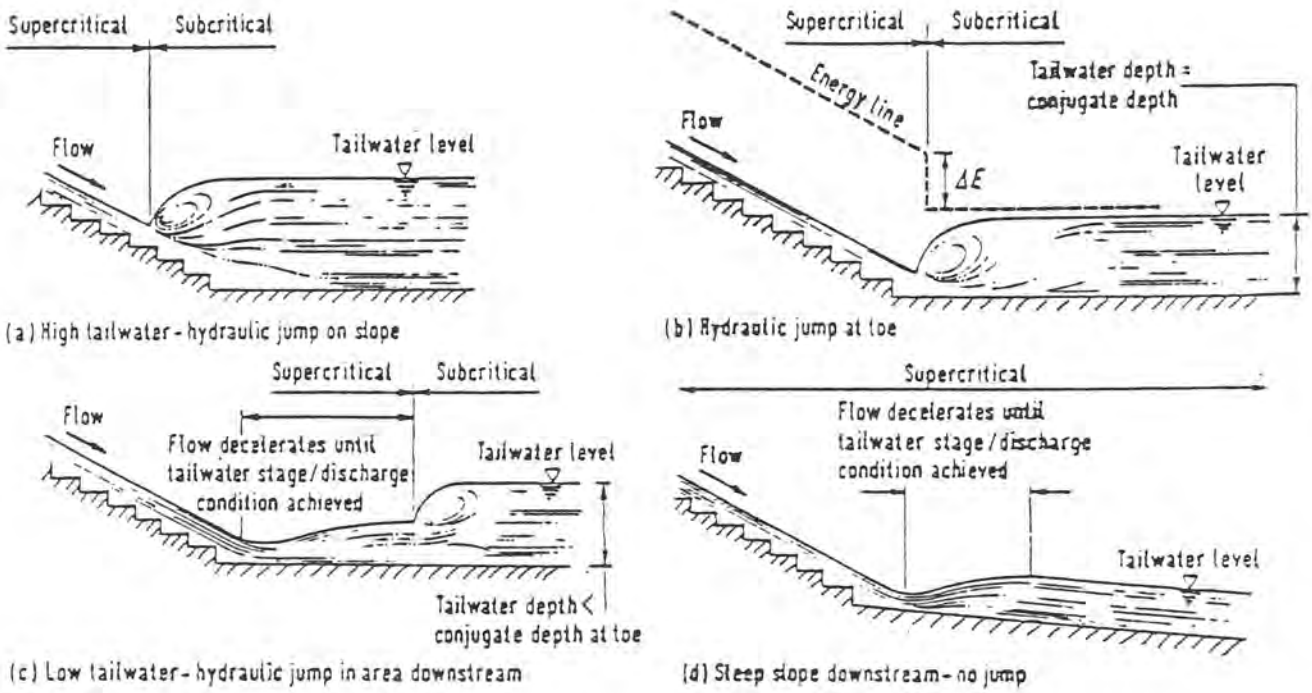
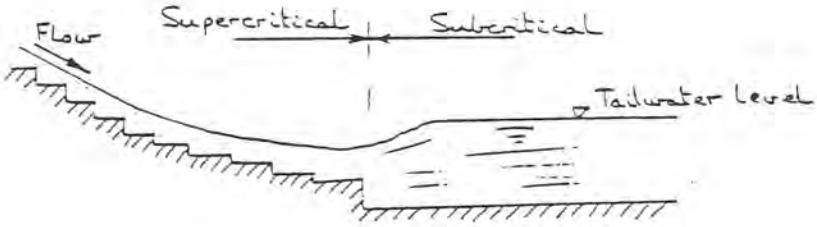


Fig 4.2: Development of Boundary Layer and Air Entrainment





(e) Surface jump

Fig 4.3: Flow Conditions at Toe

5. GEOTECHNICAL STABILITY

5.1 Introduction

The general slope of the spillway will depend on the stability of the underlying embankment fill (or existing subsoil if the spillway is constructed in a cutting). A major factor affecting the slope stability will be the effect that water movement will have on the subsoil. Water may enter the subsoil in the following ways, (illustrated in Figure 5.1).

1. Through surface cracks and other fissures. These may occur due to soil shrinkage, root holes, animal burrows etc. Seepage into these will take place as soon as the spillway comes into operation. Where deep cracks exist, these could give rise to rapid and deep, albeit localised, saturation of the adjacent subsoil.

2. By infiltration through the spillway. The rate at which this occurs will depend on the infiltration rate of the subsoil, which can vary from 100mm/hour for a sandy clay soil to very much less than 1mm/hour for a heavy clay depending on the extent of surface cracks and other fissures.

3. By seepage through the subsoil from upstream (e.g. through a dam embankment). This can be limited by a cut-off or other impermeable barrier.

Seepage into the subsoil will increase the soil moisture content and pore water pressures, leading to a reduction in soil strength. In subsoils of low permeability, a steady seepage flow may not develop while the spillway is in operation and it will be necessary to consider the rate of subsoil saturation by infiltration and the spreading of the wetted front to determine design soil moisture conditions. If flow occurs down the spillway for a long period then the subsoil may become fully saturated with a steady seepage flow developing, and this condition would need to be considered in the slope stability analysis. The underlayer beneath the stepped blocks performs the important function of assisting drainage of the subsoil both while the spillway is operating and after it has ceased. Seepage flows are likely to be small in low permeability soils and only a nominal thickness of drainage layer will be required (see Section 6.7).

In subsoils of higher permeability a steady seepage is more likely to develop when flow occurs down the spillway. Figure 5.2 shows a typical seepage flow net: water flowing down the spillway will enter the subsoil on the downstream side of the crest and exit near the toe. Gerodetti (1981) undertook some tests on a model of a rockfill dam which showed that, when overtopping started to occur, all the flow seeped into the embankment up to a prototype flow of 4.4 $m^3/s/m$. The drainage layer must be capable of passing the anticipated seepage flows. With high seepage flows, it is necessary to ensure that provision is made at the downstream end of the spillway for the seepage flows to escape either through drainage holes in the blocks or pipes through a toe block (see

Section 6.9). If the drainage layer is not capable of passing the seepage flows then the blocks are likely to be lifted up leading to failure of the spillway.

Stepped block spillways should not be constructed on high permeability granular soils (permeability in excess of about 10^{-3} m/s) due to the difficulty of passing extreme seepage discharge through the protection layer while restricting uplift pressures to acceptable limits.

Experience in the USSR has shown that in clayey soils with low saturated shear strength the slope may need to be as flat as 1 in 8. In such situations the spillway slope may need to be flatter than the general embankment slope, with the downstream end of the channel projecting beyond the toe of the dam; this has the advantage in that the unprotected toe of the dam is unlikely to be affected by high velocity flow in the tailwater channel.

5.2 Ground Investigation

It will be necessary to carry out a ground investigation before commencing the detailed design.

In some cases, information may be available from previous works on or near the site which will reduce the extent of the investigation required. A ground investigation should be carried out in accordance with BS 5930 and reference should also be made to CIRIA Special Publication 45. It should be supervised by an experienced geotechnical engineer with the principal objectives being as follows:

1. To inspect the nature of the soils beneath the proposed spillway.

2. To obtain disturbed and undisturbed samples for identification and laboratory testing.

3. To carry out in-situ tests to determine the mechanical properties of the soil.

4. To determine existing, and assess future, ground water levels.

Trial pits will normally be sufficient for obtaining this information but it may be more economic to sink boreholes for depths exceeding 3m. It is suggested that an initial exploration consisting of three or four evenly spaced trial pits should be carried out to obtain a general view of the whole site. If there is a variation in the materials encountered in these pits, then further trial pits should be excavated.

It is important to inspect the undisturbed soil fabric in the trial pits for discontinuities such as cracks or root holes which may allow water to penetrate the subsoil, both rapidly and deeply. Fissures can also cause stress concentrations, leading to local failures which gradually reduce the shear strength of the soil.

Trial pits are usually dug using a hydraulic backacter excavator and provided with support as necessary. The safety aspects of excavation of trial pits need to be considered, in particular the requirement to provide support. They should be backfilled as soon as logging, sampling and testing have been completed, because open pits are a hazard and may collapse if their sides are unsupported. Backfilling should be in layers not exceeding 300mm thickness, with each layer being compacted using the excavator bucket or other mechanical means in order to retain similar drainage properties to the adjacent soil. Poorly compacted backfill will cause settlement at the ground surface and can lead to excessive seepage flow. Extreme care must be taken when excavating trial pits on steep slopes to ensure that the slope remains stable. It may be easier and safer in some cases to excavate trial pits by hand rather than by mechanical excavator.

The depth of trial pits will depend on the materials encountered and preliminary design considerations. In general a depth of 2m should be sufficient, but if weak, or very permeable materials are encountered it may be necessary to go deeper. The location of all trial pits and boreholes should be recorded on a plan and the faces of all trial pits should be photographed in colour.

5.3 Testing

5.3.1 General index tests

The following tests may be carried out to ascertain the general nature of the subsoil material. Sufficient number of tests should be undertaken to provide general information on the subsoil if different horizons exist, or to identify unsuitable material (e.g. highly organic material):

1. Vane shear test, to give the undrained shear strength, details of which are given in BS 5930.

2. Natural moisture content, void ratio and bulk density.

3. Liquid and Plastic Limits (for fine-grained soils).

4. Particle size analysis (particularly for coarse-grained soils).

5. Simple in-situ infiltration test at the proposed formation level using a ring infiltrometer. This will be of interest when considering the depth to which water may infiltrate the subsoil during operation of the spillway.

Details of tests 2, 3 and 4, which are carried out in the laboratory, are given in BS 1377.

Details of infiltration tests can be found in agronomic or soil physics (as opposed to geotechnical) reference books. The most common form of ring infiltrometer comprises two concentric metal cylinders about 250mm and 400mm in diameter driven into the soil -

infiltration being measured by ponding water in both and then measuring the rate of fall in the inner. It should be noted that the shear strengths given by test 1 are only approximate and should not be used for design purposes.

5.3.2 Tests required for design of spillway

These tests are necessary to determine the soil parameters which will be required when designing the spillway:

1. Testing by shear box or triaxial apparatus to determine the shear strength parameters, either:

   Undrained shear strength, C_u, applicable to cohesive soils, to be used when considering slope stability in terms of a total stress analysis and will also be of interest when considering the provision of mechanical restraints. Tests should be carried out on undisturbed samples obtained as described in BS 1377 both at the natural moisture content and on saturated samples to represent the state of the materials under normal conditions and when the spillway is operating. Care should be taken in assessment of the tests as the quality of sampling, the size and direction of sampling, the presence of fissures etc can have substantial effects on the results and a sufficient number of tests should be carried out. The undrained shear strength is also dependent on the moisture content of the soil and will vary substantially with changes in this, thus leading to difficulties in selecting a suitable value for use in the stability analysis.

   Drained (effective stress) shear strength parameters, c' and ϕ'. These parameters allow a potentially more accurate assessment of stability to be made, being independent of moisture content and less affected by the processes above, but require more complex and lengthy (and therefore expensive) testing to be carried out.

   The use of either test, especially that for drained shear strength, requires guidance from an experienced geotechnical engineer both in the choice and in the interpretation of the test results. Appropriate advice should be sought in all cases other than the most straightforward of slopes.

2. Tests for sulphate content of the ground water if ground water levels are near the surface. Sulphate content tests are necessary as if sulphates are present they may attack the concrete. Reference should be made to BRE Digest 250 to see if sulphate resisting cement should be used in the concrete.

   Details of these tests may be found in BS 1377.

5.4 Design

5.4.1 Slope stability

The stability of the slope on which the spillway is to be constructed should be investigated by one of the standard methods of slope stability analysis and an adequate factor of safety shown to exist. The foundation should also be considered in the analysis in addition to the slope itself. The stability of any side slopes to the spillway channel should also be investigated. The stability should be investigated in four ways:

1. The stability during construction of the spillway.

2. The stability under normal conditions when the spillway is not operating.

3. The stability during operation when the subsoil has become partially, or fully, saturated.

4. The stability immediately following operation as the subsoil drains.

Construction of a spillway may involve excavation of an existing slope. This excavation could cause instability and this should be considered at the design stage. No excavation which would make the slope steeper, even during construction, should be carried out without considering its stability.

The stability of the slope when the spillway is not operating can be readily calculated using the best assessment of the undrained strengths applicable to the material in the in-situ (unsaturated) condition.

The stability of the slope during and after operation of the spillway will depend on the extent to which the subsoil has become saturated. The designer should consider this on the basis of general trial pit information as well as from the more rational viewpoint of the infiltration characteristics of the soil and the duration of flow down the spillway.

The designer should consider the additional loading due to the weight of the blocks and the water flowing over them.

The stability analyses are best carried out in terms of effective stress using the parameters c' and ϕ' making a realistic assumption of pore pressure. The method is more complex but should offer some greater certainties in the assessment of the stability of the spillway both in the long term and during operation of the spillway. This approach should only be carried out by or in conjunction with an experienced geotechnical engineer. Care must be taken with the measured parameters as recent work has shown that the true value of c' may be lower than that obtained by current laboratory testing practice and the use of effective stress analyses in this situation may overestimate the stability of the slope.

The stability analyses could alternatively be carried out in terms of a total stress approach, but care must be taken to consider the undrained strengths at the correct in-situ moisture content. Normally an analysis based on the fully-saturated material would be applicable.

5.4.2 Drainage

The provision of a free-draining underlayer beneath a stepped block spillway is an important requirement. This underlayer performs the following functions:

1. underdrain which is free draining to collect seepage flows beneath the blocks, facilitate their evacuation via the drainage holes in the blocks, and hence prevent the build-up of high uplift forces;

2. filter to retain the formation subsoil from movement due to seepage flow emerging from the subsoil;

3. erosion protection medium to protect the subsoil from erosion due to high downslope seepage velocity in the underdrain;

4. regulating layer to provide an even foundation for the blocks.

It has been normal practice in the Russian installations for the underlayer to consist of two or three separate layers of granular material. Geotextiles are not widely available in Russia and there is no record of them having been used in an underlayer. They are, however, now widely used in the UK for filtration, erosion control and separation functions while granular materials are generally preferred for regulating and drainage. Different features of geotextile and granular materials as underlayers are given in Hemphill and Bramley, and these are reproduced in Table 5.1. A combination of granular drainage layer(s) and a geotextile filter may provide the most effective and economic solution for the underlayer to a stepped-block spillway. If a geotextile is proposed for use immediately beneath the stepped blocks then consideration should be given to (a) the effect of friction between the blocks and geotextile on the overall stability of the spillway and (b) that the opening size of the geotextile is sufficient to enable the underlayer to drain adequately through the holes in the stepped blocks. The principal advantage of using a geotextile in this situation would be to prevent loss of material from the underdrain. Detailed guidelines for the design of the underdrain are given in Section 6.7.

Table 5.1 Features of geotextile and granular material as underlayers

Geotextile	Granular material
Advantages	
In-plane tensile strength	Self-healing in some circumstances
Limited thickness	Generally very durable
	Deformable, retaining good surface contact above and below
	Relatively easy to repair
Disadvantages	
Some uncertainty over long-term behaviour	Careful control needed to achieve specified grading and thickness
Edges must be carefully protected	Compaction difficult on steep side slopes
Easy to damage; difficult to repair	Control of construction difficult underwater
Careful design and installation needed to accommodate settlement or uneven formation	

5.4.3 Settlement

Settlement of an embankment could cause gradual displacement of the blocks so that they no longer function correctly when subjected to flow. Settlement could also result from incorrect design of the underlayer/drainage holes with flow removing material from the underlayer leading to settlement and block displacement.

A new embankment compacted by earthmoving plant will undergo some settlement after the completion of construction due to the consolidation of the foundation and fill materials under the load of the embankment. The amount of further settlement depends on many factors including the type of fill and foundation materials, the length of the construction period and the effectiveness of the compaction during construction. Any allowance in the fill height for settlement should be determined by an experienced geotechnical engineer.

The period of further settlement can take many years and thus some settlement can be expected unless the embankment is known to be of a substantial age, or records show that ongoing settlement is minimal. If substantial settlement is anticipated then a stepped-block spillway should not be used unless it is sufficiently flexible to accommodate the movement expected.

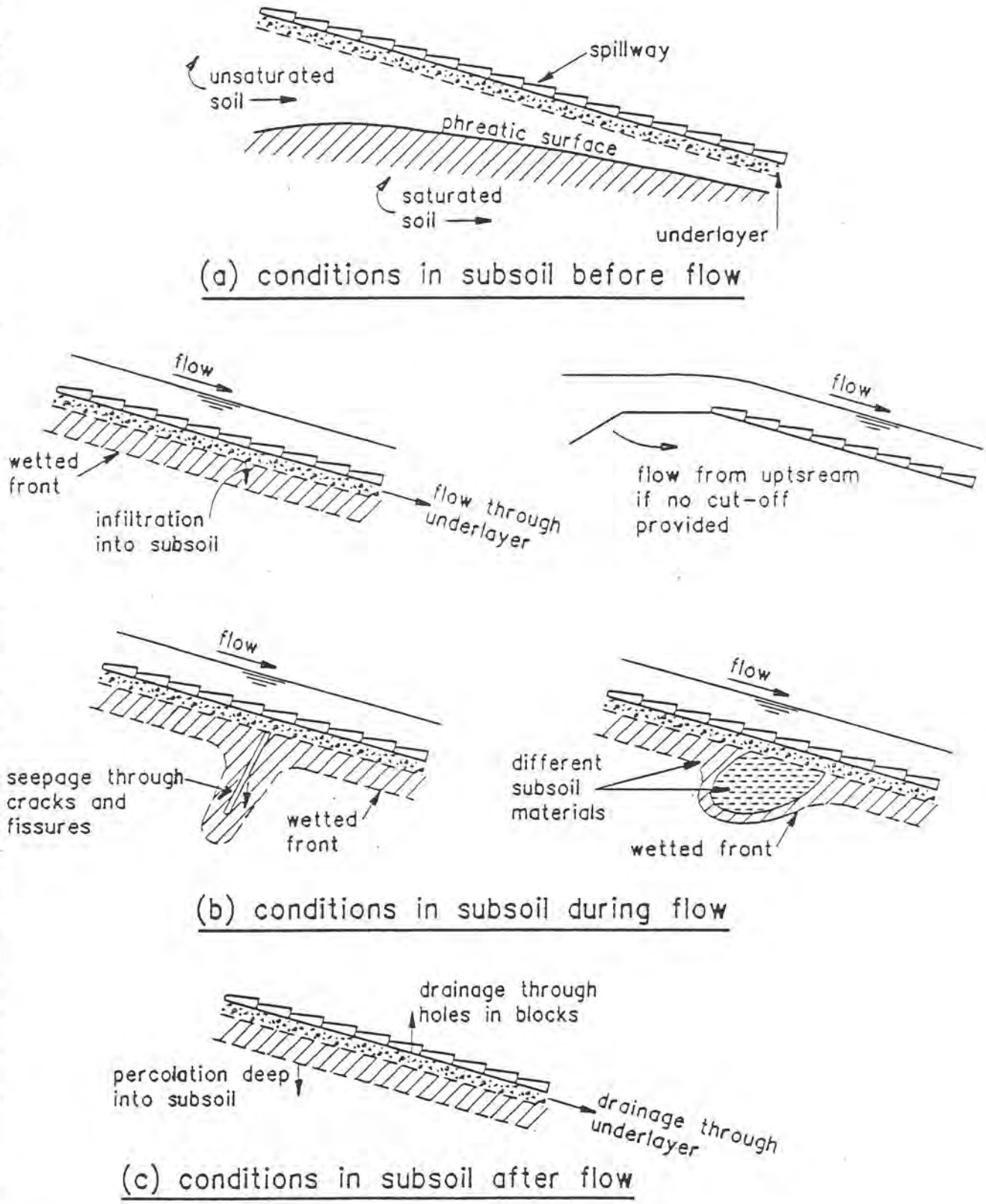


Fig 5.1 Seepage Flow and Infilltration

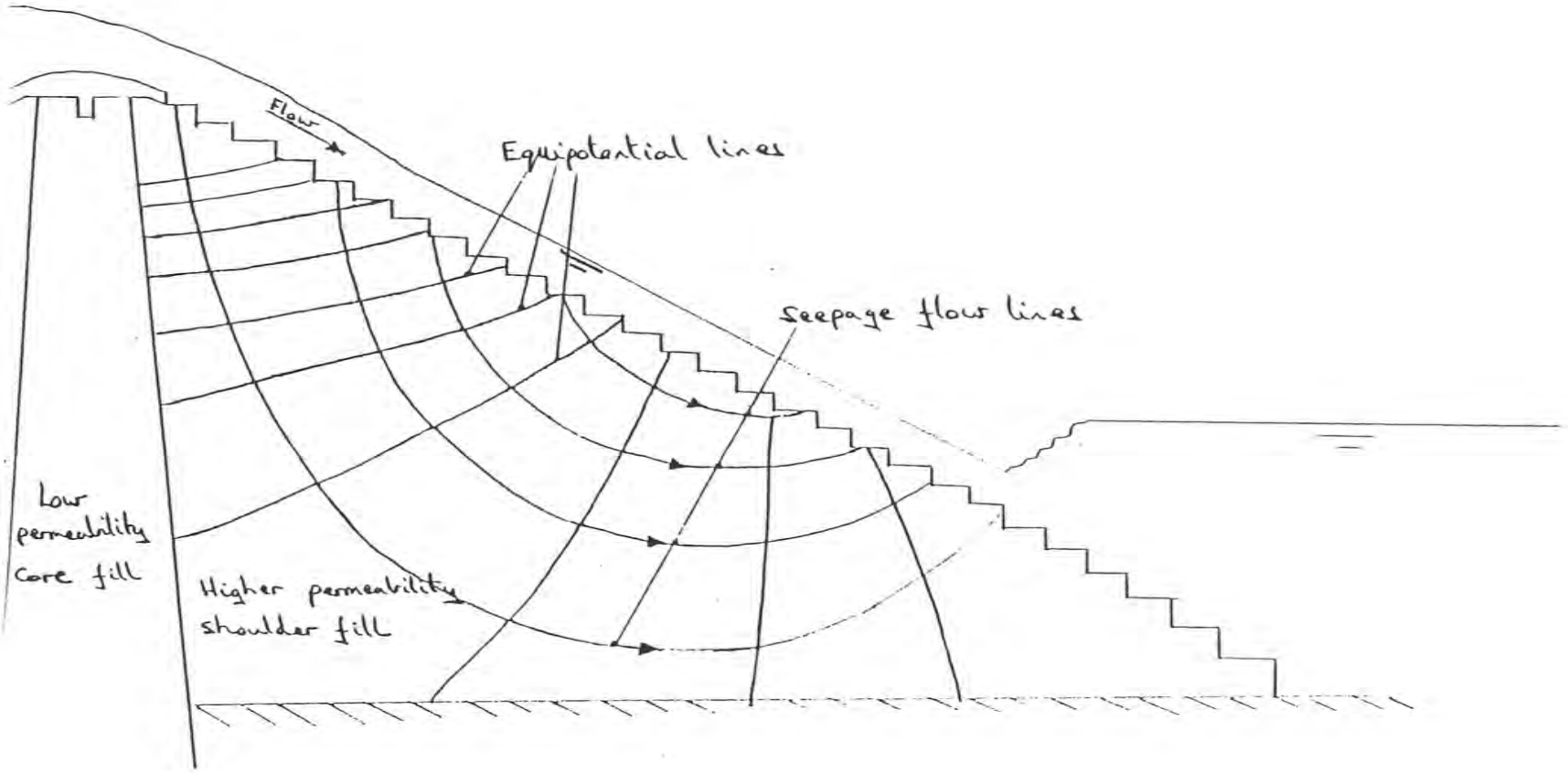


Fig 5.2 : Typical seepage flow net

6. DETAILED DESIGN

The design of a stepped-block spillway requires careful attention to detail, since any spillway is only as effective as its weakest point. This section provides information on various design details that need to be considered when preparing the construction drawings.

6.1 General Arrangement

The general arrangement of the spillway should take into account:

1. geometric requirement for satisfactory flow path;

2. constructional considerations, in particular the use of the spillway for temporary works at different stages of construction;

3. access requirements for monitoring and maintenance, and prevention of access of heavy plant onto the spillway if the blocks will not be designed to take high surface loading;

4. access across the spillway for foot and/or vehicular traffic.

Among the detailed points to be considered are:

Plan

The alignment of the spillway in plan should be straight from the crest to the toe. Change of direction in plan or change in the width will cause an uneven distribution of flow. In supercritical flow, this can also cause surface disturbances and wave patterns to be set up which perpetuate downstream. Such changes should therefore be avoided, or special consideration (e.g. a physical model study) should be given to identifying and designing for the actual flow conditions.

Slope

The slope of the spillway will be determined by the stability of the underlying subsoil (see Section 5) which may be either embankment fill or original ground. The slope may vary along its length, in particular it may be flatter at the downstream end to facilitate the transition to subcritical flow. Further details of slopes in the toe region are given in Section 6.9.

Width

The width of the spillway will be determined by the design discharge intensity which will be based on consideration of the economics of the following:

(a) block size;

(b) energy dissipation at tailwater and erodibility of channel downstream;

(c) distance between weir crest level and top of embankment.

It must be understood that should the protection fail, the subsequent rate of erosion of the subsoil (other factors being equal) will be greater, the higher the discharge intensity.

Access across spillway

Access across the spillway can be achieved by

(a) a bridge or box culvert. Any constriction in flow should be located at the spillway crest within the subcritical flow region;

(b) restricted low-flow crossing, e.g. a water splash for vehicles, with the sides ramped down at acceptable slopes (say, 1 vertical to 7 horizontal) into the spillway forebay. Dry pedestrian access across the forebay can be accommodated by 'stepping stones'.

6.2 Block shape

As mentioned in Section 2.1 three different shapes of block have been investigated and all shown to be hydrodynamically stable. Each type of block has certain advantages and disadvantages, and these are summarised in Table 6.1.

Table 6.1 Features of different types of block

Type of block	Advantages	Disadvantages
a) Overlapping and wedge-shaped.	Restricts vertical movement and provides greater stability. Drainage holes can be protected by the overlapping joints and are less liable to become clogged.	Most difficult type to manufacture. Most difficult type to replace.
b) butt-jointed and wedge shaped.	Easy to replace damaged blocks. Easier to construct than overlapping and wedge-shaped.	Possible vertical movement of blocks. Drainage holes can become clogged.

Type of block	Advantages	Disadvantages
c) overlapping slabs.	Simple to manufacture. Better frictional resistance between blocks and underlayer.	Difficult to construct because stepped foundation required. May require more reinforcement to resist bending moments. Drainage holes can become clogged.

6.3 Block size

Laboratory experiments were conducted by Grinchuk and Pravdivets (1977) and Pravdivets et al (1980) to determine the stability of stepped blocks. The factor of safety against sliding was determined from $F_s = \frac{N}{F_o} \tan \phi$ where

N is the minimum value of the normal force between the underside of the block and the underlayer, F is the friction force on the underside of the block resisting sliding and ϕ is the angle of friction between the block and underlayer, with each block being fully independent and deriving no restraint from adjacent blocks.

The laboratory data was used to derive the curves shown on Figure 6.1a relating average block thickness, t_a, to discharge intensity, q, for various spillway slopes ($s = \sin \theta$). t_a is defined as the mean thickness of the assembled blocks i.e. the total volume of the blocks divided by the area of the spillway normal to the slope. These curves are based on a factor of safety of 1.5, a concrete density of 2.4 t/m^3 and typical ϕ values for clay with slopes of $s \leq 0.2$ and for granular materials with a slope of $s = 0.3$.

The two sets of curves in Figure 6.1a are fairly similar, but differ slightly in terms of their slopes and values of minimum block thickness. Only two points were determined for each slope, and so a straight line was drawn between these points. If Froudian scaling is assumed, however, the relationship between block thickness and discharge should be non-linear, particularly at low discharges where block thickness should theoretically tend to zero. Figure 6.1b shows Froude similarity curves at relatively low discharges which have been determined from the Russian laboratory results at higher discharges. A minimum average block thickness of 0.1m has been assumed, principally on the grounds of handling of the blocks and on concrete durability. It is theoretically possible for thinner blocks to be used with high grade precast concrete and favourable climatic conditions.

It is recommended that Fig. 6.1b is used to determine the minimum block thickness for discharges below $40 m^3/s/m$, with the solid lines in Fig. 6.1a being used for higher discharges.

The upper surface geometry of a stepped-block spillway is defined in terms of step height and step length L (as installed, and as distinct from overall block length). For hydrodynamic stability, the ratio L/Δ must be within the limits:

$$4 < L/\Delta < 6 \qquad \text{.....(6.1)}$$

It is particularly important that L/Δ is not less than 4, since laboratory tests have shown that the blocks become unstable at about 3.5, with the flow failing to land on the next block downstream. Further details of these laboratory tests are given in Appendix A2.

Stability is not critically dependent on the underside geometry of the block, the overlap between blocks or the width of the block. To maintain reasonable flexibility, block width should not be substantially greater than block length, although where settlement is not considered to be a problem, block width might be increased in order to increase block weight and prevent vandalism.

Although the minimum recommended average thickness is 100mm, parts of the wedge shaped blocks will be thinner than this, in particular the overlapping lip if one is used. For mass or lightly-reinforced blocks, a minimum concrete thickness of 75mm is recommended from practical considerations of fabrication, handling and durability. Heavily reinforced blocks will need to be thicker, so that adequate cover is provided to the reinforcement. In addition the aspect ratio (maximum length to average thickness) should not be greater than 4 unless the block is reinforced. The dimensions of a typical wedge shaped and overlapping block are given on Figure 6.2.

Once the minimum size of block has been determined, consideration can be given to using a larger size of block. There are a number of advantages and disadvantages of varying sizes of block and these are listed in Table 6.2.

Table 6.2 Factors affecting choice of block size

Size of block	Advantages	Disadvantages
Large blocks.	Greater discharge capacity possible; Less prone to vandalism.	More difficult to construct; More prone to settlement problems; Greater quantities of concrete and reinforcement required.
Small blocks.	Easier to manufacture and construct; Greater flexibility.	More liable to be damaged; Larger number of blocks required.

6.4 Hydraulic roughness

It will be necessary to determine the hydraulic roughness in order to calculate the depth of water in the spillway for (a) design of the sides and (b) estimation of energy dissipation in the spillway chute upstream of the tailwater together with the flow conditions entering the tailwater.

The design curves in Fig. 6.3 have been derived from model tests undertaken in Russia for blocks with $4 < L/\Delta < 6$. These curves can be used to estimate the unaerated flow depth d' at any point on the spillway, both in the upper region of gradually varied flow and, on a long spillway, in the lower region of uniform flow.

The curves are valid for values of $d' \geqslant 2\Delta$. For a given spillway slope s, upstream head H and discharge intensity q, the value of the function $\bar{\Phi}$ in the equation

$$d' = \frac{q}{\bar{\Phi}\,[2g(p+H)]^{\frac{1}{2}}} \qquad \text{.....(6.2)}$$

may be obtained for any location on the spillway. The vertical distance p below the crest is measured to the water surface. The design curves for stepped spillways are indicated by broken lines, while the continuous curves represent data for smooth concrete spillways.

A series of gradually-varied flow profiles should be calculated for the chosen spillway geometry over a range of flow conditions from low to maximum discharge using Fig. 6.3. The results should be reviewed to check the general acceptability of:

(a) maximum velocity head on the spillway chute;

(b) energy loss and power dissipation in the tailwater;

(c) velocity in the tailwater.

The discharge intensity may then be modified by altering the spillway geometry and repeating the process.

6.5 Drainage holes

The drainage holes in the stepped blocks make an important contribution to their good performance by helping to reduce uplift pressures on the lower surfaces and evacuate seepage flows from the underdrain. The holes should be located in the low pressure separation zone which forms immediately downstream of each step. Many laboratory tests, including those at Salford have been conducted on blocks with different hole sizes, and the general conclusion is that the total area of holes should be between 2.5% and 5% of the exposed block area in plan. These holes should all be located within a distance of 2Δ downstream of a step where Δ is the step height.

The diameter of the holes is restricted by the need to prevent removal of material from the underdrain, although this would be less of a problem if a geotextile were used between the blocks and underlayer (see Section 5.4.2). Drainage holes in butt-jointed blocks and overlapping slabs are normally circular, and exposed on the horizontal face, where they can easily become blocked with leaves etc. Overlapping and wedge-shaped blocks can, however, have drainage vents formed in the downstream face (see Fig. 2.2). Although this reduces the likelihood of them becoming clogged, only a relatively small drainage area can be provided in this way, and it may be necessary to include a row of circular holes in the slab in addition to such vents.

6.6 Spillway sides

The preferred spillway cross-section for economy of cost and ease of construction is trapezoidal, with stepped blocks being laid on the sloping sides. The side slopes depend on the geotechnical properties of the subsoil (see Section 5.4.1): typical side slopes are 1 vertical to 3 horizontal. The height of the sides should be based on the air-bulked flow depth (shown on the Salford model to be between 130% and 160% of the unaerated or 'blackwater' depth determined using Fig. 6.3). An additional allowance of at least 0.5m should be made for freeboard.

Typical side details are shown in Figure 6.4. Care should be taken over the design of the invert/side joint. If joints are used at the bottom of the side slopes then it will be necessary to use half blocks every other row in order to construct the spillway in stretcher bond. The Salford model tests showed the half blocks to be less stable than full-size ones and their use is not recommended for discharges in excess of 5 m^2/s where special angle blocks should be cast as shown in Fig. 6.4(a). For lower discharge intensities there can be a joint at the bottom of the side slope, with the blocks on the sloping side being restrained by a shear bar and the joint being covered by an in-situ concrete fillet.

Precast or in-situ concrete retaining walls could be used for the channel sides, but care needs to be taken to fill in any gaps between the blocks and side walls with in-situ concrete.

6.7 Underlayer

The functions of the underlayer were described in Section 2.1.1. It will normally contain one or more layers of granular material for drainage and a granular or geotextile filter.

6.7.1 Drainage layer design

Grinchuk and Pravdivets (1977) and Pravdivets and Slissky (1981) suggest the following rules for the design of the drainage layers:

$$5 > \frac{D_{15}}{d_{85}} > 4 \qquad \text{.....(6.3)}$$

where D_{15} is the size not exceeded by 15% by weight of particles in the upper layer and d_{85} is the size not exceeded by 85% of particles in the lower layer. The thickness of each layer should be between 3 and 4 times its own d_{85}, but not less than 0.2m.

The size of drainage holes in the blocks must be small enough to prevent the loss of material from the underdrain, i.e. no larger than d_{85} of the upper drainage layer.

6.7.2 Filter design

The function of the filter is to prevent migration of the subsoil particles while allowing movement of water across the filter-subsoil boundary. Either geotextile or granular filters may be used.

(a) Geotextile filters

Present understanding of the physical processes of filtration by geotextiles is incomplete. Filter performance depends on the properties of the base soil, notably:

1. Characteristic particle size, generally expressed as the sieve size, D_{nb}, that n% by weight of the base soil particles are smaller than.

2. Uniformity, generally expressed as a uniformity coefficient ($U = D_{60}/D_{10}$).

3. Whether the soil is compact or loose.

4. Whether the soil is cohesive or non-cohesive.

5. Permeability, k_b, of the soil in the direction of flow.

and the geotextile, principally:

1. Characteristic opening size, generally expressed as the pore size, O_n, that n% of the pores are smaller than.

2. Geotextile fabric type, whether woven or non-woven.

3. Permeability, generally expressed in terms of permittivity, ψ. (Permittivity can be measured directly in a constant head permeability test, and expresses the rate of flow through the geotextile per unit area per unit head. Permeability is equal to permittivity multiplied by a nominal fabric thickness).

Filter properties are generally specified in terms of fabric type, pore size and permeability. There are several different methods for designing geotextile filters in current use and the designer should consult references such as PIANC (1987a), van Zanten (1986) and manufacturers' source data. Filter rules are also given in Hemphill and Bramley (1989) and these are included as Appendix A3 in this Report.

(b) Granular filters

If a granular filter layer is proposed, then its grading must be compatible with the adjacent drainage layer using the rules in Section 6.7.1.

For relatively uniformly graded soils (say with uniformity coefficient U < 5):

$$D_{50f} < 5D_{50b} \qquad \text{.....(6.4)}$$

where suffix f relates to the filter.

For well-graded base soils (say U > 10):

$$D_{15f} < 5D_{85b} \qquad \text{.....(6.5)}$$

but to minimise segregation, the two grading curves should not be too far apart.

Thus:

$$D_{50f} < 25D_{50b} \qquad \text{.....(6.6)}$$

The grading curve of the filter should be approximately parallel to that of the base soil.

The permeability of the base soil and the filter is determined by the smaller particle sizes. For the filter to be substantially more permeable than the subsoil:

$$D_{15f} > 5D_{15b} \qquad \text{.....(6.7)}$$

CUR-VB (1984) suggest that in order to prevent blockage, the minimum size of fine filter particles, D_{5f}, should be greater than 75 μm (0.075mm).

Further information on granular filters can be found in Stephenson (1979) and de Graaw et al (1983).

6.8 Crest details

The upstream end of the crest should be terminated in a low velocity area so as to avoid the risk of erosion by the approaching flow from upstream. The design of the crest should also discourage excessive seepage flow from upstream into the underlayer, and a cut-off, consisting of concrete, clay or other impermeable membrane, may be incorporated.

The crest should be formed of an in-situ concrete overflow weir. The discharge characteristics will depend on the upstream slope and the shape of the crest. The upstream face may need to be protected by rip-rap, concrete slabbing, gabion mattresses or other materials to prevent erosion by flow approaching the crest. A suitable slope on the upstream side of the crest is 1:2.5 or 1:3.

Where a purpose-shaped weir is constructed to form a control, the discharge coefficient, C, in the general weir flow equation

$$q = C H^{3/2} \qquad \text{.....(6.8)}$$

is generally between 1.7 and 2.0 in S.I. units: values are given in standard hydraulics reference books such as Chow, Henderson or King and Brater.

Access across the crest should be provided as discussed in section 6.1.4.

The crest should have a smooth curve downstream of the weir, avoiding any sharp changes in slope. Negative pressures could occur downstream of any such changes in slope due to separation of the flow, leading to unacceptable uplift forces on the first rows of blocks.

Typical crest details are shown in Figure 6.5. It is important that the downstream end of the crest block should be adequately joined to the first row of blocks in the spillway chute. The crest should overlap the first row of blocks to shield the leading edges of the blocks from flow impact. The height of the step should be similar to that on blocks down the spillway.

6.9 Toe Details

The spillway toe and tailwater transition requires careful design so that the specific energy of the flow is dissipated without adversely affecting the stability of the spillway blocks or causing excessive scour downstream of the spillway. The extent of the tailwater toe works is affected by (a) the range of tailwater levels, (b) the erodibility of the foundation materials, (c) the channel slope and (d) the discharge intensity (since this affects the energy of incoming flow). As explained in Section 4, provision for energy dissipation at the tailwater transition (as opposed to the stability of the spillway protection under free flow conditions) is likely to be the principal factor influencing the choice of design discharge intensity. Three types of toe design have been studied and these are described in the following sections:

6.9.1 Stepped blocks

Tests in the hydraulic model at Salford showed that failure of stepped blocks was more probable under a hydraulic jump than under free flow conditions (see Appendix A2.2). This is because (a) the high turbulence in a jump contains flow travelling in a reverse direction, producing large forces on the sides of the steps, (b) sudden pressure fluctuations may tend to lift the blocks vertically and (c) the low pressure zone downstream of each step ceases to exist under a jump and there could be a build-up of seepage flows underneath the blocks. The stability of stepped blocks under a hydraulic jump can be increased by using thicker blocks. The tests showed the blocks to be more stable on flat

slopes and Figure 6.6 shows design curves for stepped blocks under a hydraulic jump for different slopes. These curves have been produced by Froude scaling from the Salford model for the situation where no block movement occurs. They include a factor of safety of 2 on a 1 in 6 slope increasing to 4 on a slope of 1 in 2.5.

6.9.2 Deflector block

The toe design that has been used successfully in Russian installations incorporates a large deflector block securely founded at the downstream end of the sloping chute. This design was confirmed to be adequate when tested on the Salford model. This block directs the flow away from the bed and causes it to form a surface jump. In order to avoid a plunging type of hydraulic jump, it is necessary for the slope of the spillway channel to be flattened to a gradient not steeper than 1 vertical : 6 horizontal for a distance upstream of the toe of about 8 times the flow depth at maximum discharge. The upper part of the spillway may be constructed at a steeper slope (see section 5) with a smooth transition to a 1 in 6 slope near the toe. Typical details are illustrated in Figure 6.7.

Minimum dimensions for the deflector block are shown on Figure 6.7 The downstream lip of the block must not be above the minimum tailwater at the lowest discharge. The height 'a' of this lip above the invert of the channel needs to be chosen so as to promote the formation of a surface jump. Laboratory tests in the Russia have shown that dimension 'a' is related to the depth 'y' and flow velocity 'V' at the downstream end of the chute by:

$$a = 2y\ (F_r^{2/3} - 1) \qquad \text{.....(6.9)}$$

where F_r is the Froude number given by

$$F_r = \frac{V}{(gy)^{1/2}} \qquad \text{.....(6.10)}$$

The maximum depth of tailwater, y_m, required above the channel invert for satisfactory operation may be determined from

$$y_m = 0.93\ (a + 1.7\ y_c) \qquad \text{.....(6.11)}$$

where y_c is the critical depth, which may be determined from:

$$y_c = (q^2/g)^{1/3} \qquad \text{.....(6.12)}$$

The design of the deflector block must allow for drainage of the underlayer beneath the chute spillway. If the foundation consists of rock, the deflector block can be cast directly on the rock, and no further protection is likely to be required downstream. If, however, the foundation is erodible the block should be constructed on piles or other supporting structure with a cut-off constructed of sheet piles or concrete being provided to prevent undermining of the spillway. Downstream of the deflector block, the channel should be protected by a flexible revetment such as

conventional parallel-sided concrete blocks, gabion mattresses or rip-rap. A special bevelled block, which appears to be more stable than stepped blocks in turbulent flow, is being developed in Russia for use downstream of a deflector block.

The following procedure is suggested for the design of a deflector block:

1. Establish the relationship between tailwater depth, y_t, and discharge, q, for the full range of flows.

2. Check that a surface jump will form at small discharges (say 20% of maximum discharge): determine the height of the deflector block, a, using equation (6.9) and the corresponding value of y_m from equation (6.11). A surface jump will form if:

$$y_m > y_t > a \qquad \text{.....(6.13)}$$

   If necessary, the step height may be adjusted so that a surface jump does form.

3. Check that the tailwater depth at higher discharges is adequate for a surface jump to form with the same step height, a.

This design procedure is illustrated in Fig. 6.8.

Alternatively, if, at high discharges the tailwater level is so high that a hydraulic jump could form on the stepped blocks above the deflector block, then the stepped blocks in this region could be increased in size using Fig. 6.6.

6.9.3 Stilling basin

A third option for the design of the toe is to construct a conventional in-situ concrete stilling basin with stepped blocks only being used above the maximum tailwater level so that no jump could form on them. The spillway chute could have either a stepped surface as shown in Figure 6.9 or a smooth one. In most circumstances it is likely that a conventional stilling basin would be more expensive than the other two options for toe design described above. Adequate provision must be made for drainage of the underlayer if an in-situ concrete stilling basin is used. Drainage pipes should not discharge into the area where the jump is strongest, since pressure pulses could travel up the drainage layer and destabilise the blocks.

6.10 Shear restraint

If the blocks are laid with gaps between them, then it is probable that they would all slide down against one another, particularly if they are laid on a geotextile with a low coefficient of friction. Resistance to sliding could be improved by the provision of mechanical restraint between the stepped blocks and the subsoil.

This shear restraint could take the form of an in-situ concrete key or steel bars. If a concrete key is used then provision must be made for drainage of the underlayer by installing pipes through the concrete key.

If steel bars are to be used, these should be either galvanised reinforcing bars or stainless steel pins, driven through holes in the stepped blocks. If any of the holes provided for drainage purposes are used, then it must be checked that sufficient area is still available for drainage in accordance with the criteria in Section 6.5. Holes used for shear restraint should preferably be in the low pressure zone within a distance of 2Δ downstream of a step: if they are in excess of this distance, in the high pressure zone, then care must be taken to ensure that the steel pin is driven in flush with the surface of the block. A pin left proud in this zone would generate a stagnation pressure and increase the downwards pressure difference.

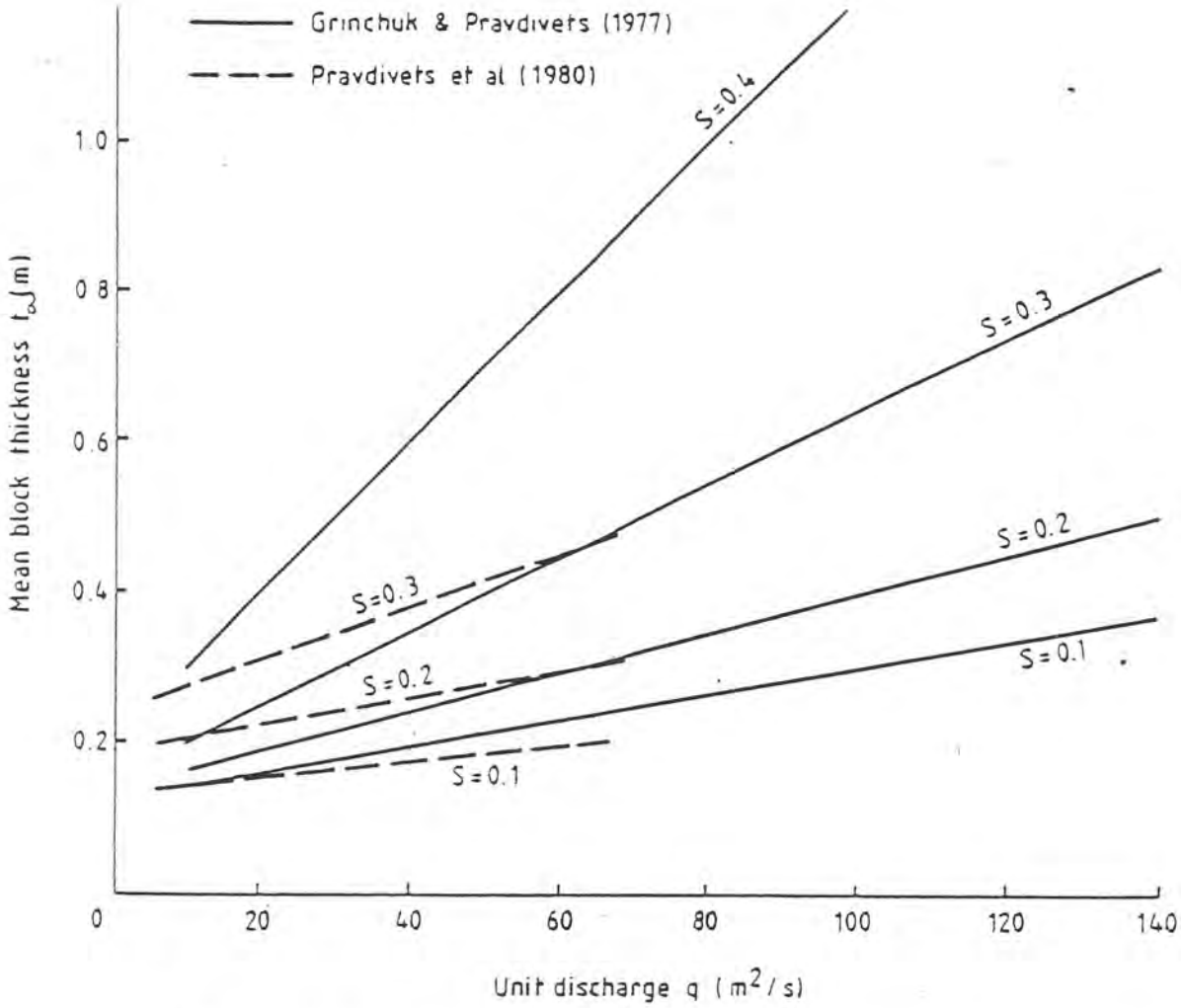
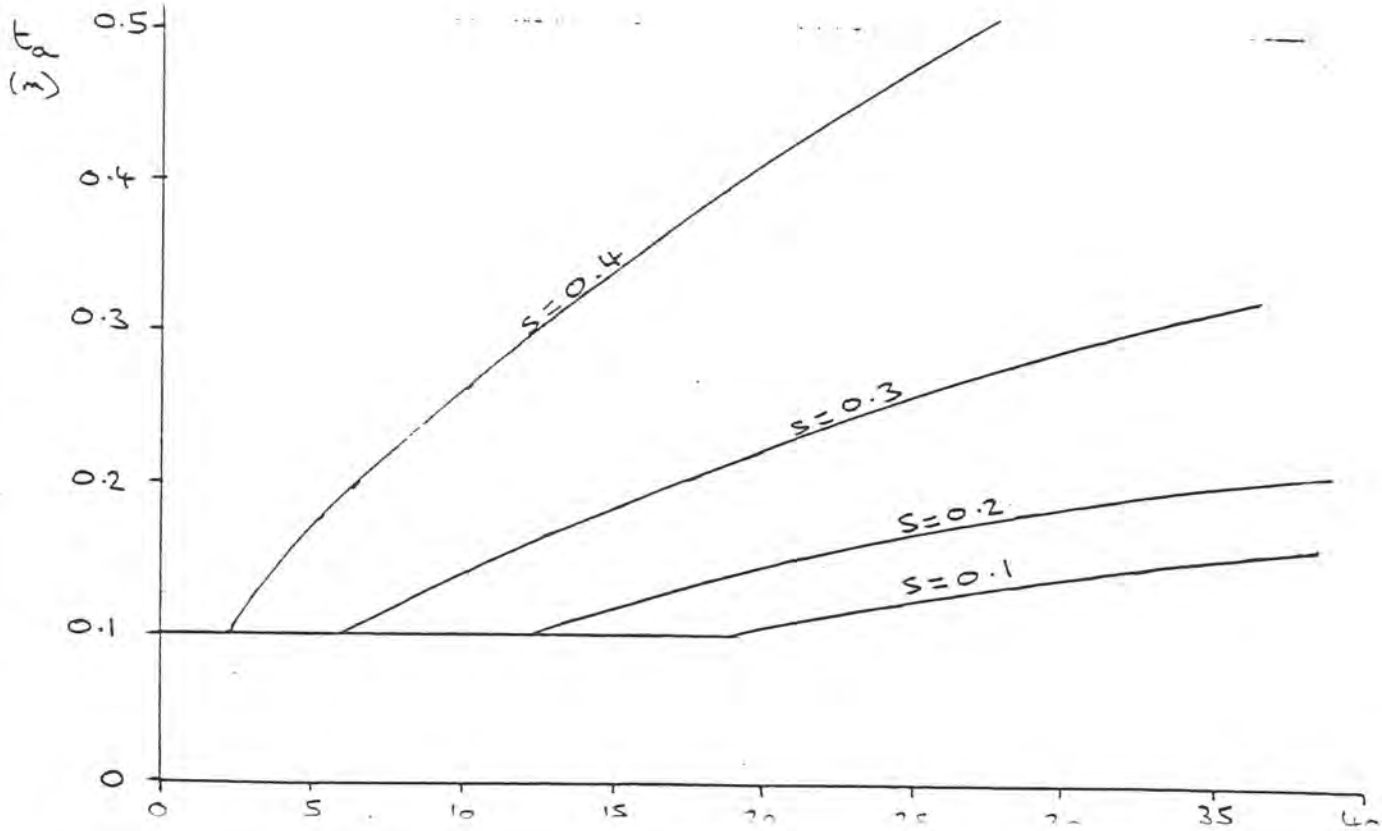


Fig 6.1 a: Design curves for block thickness (Russian)



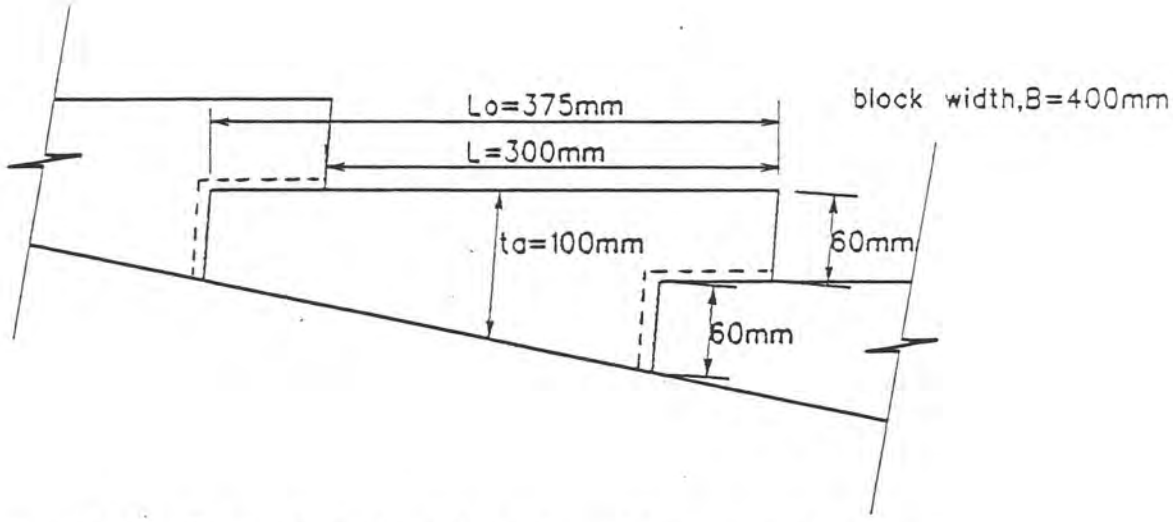
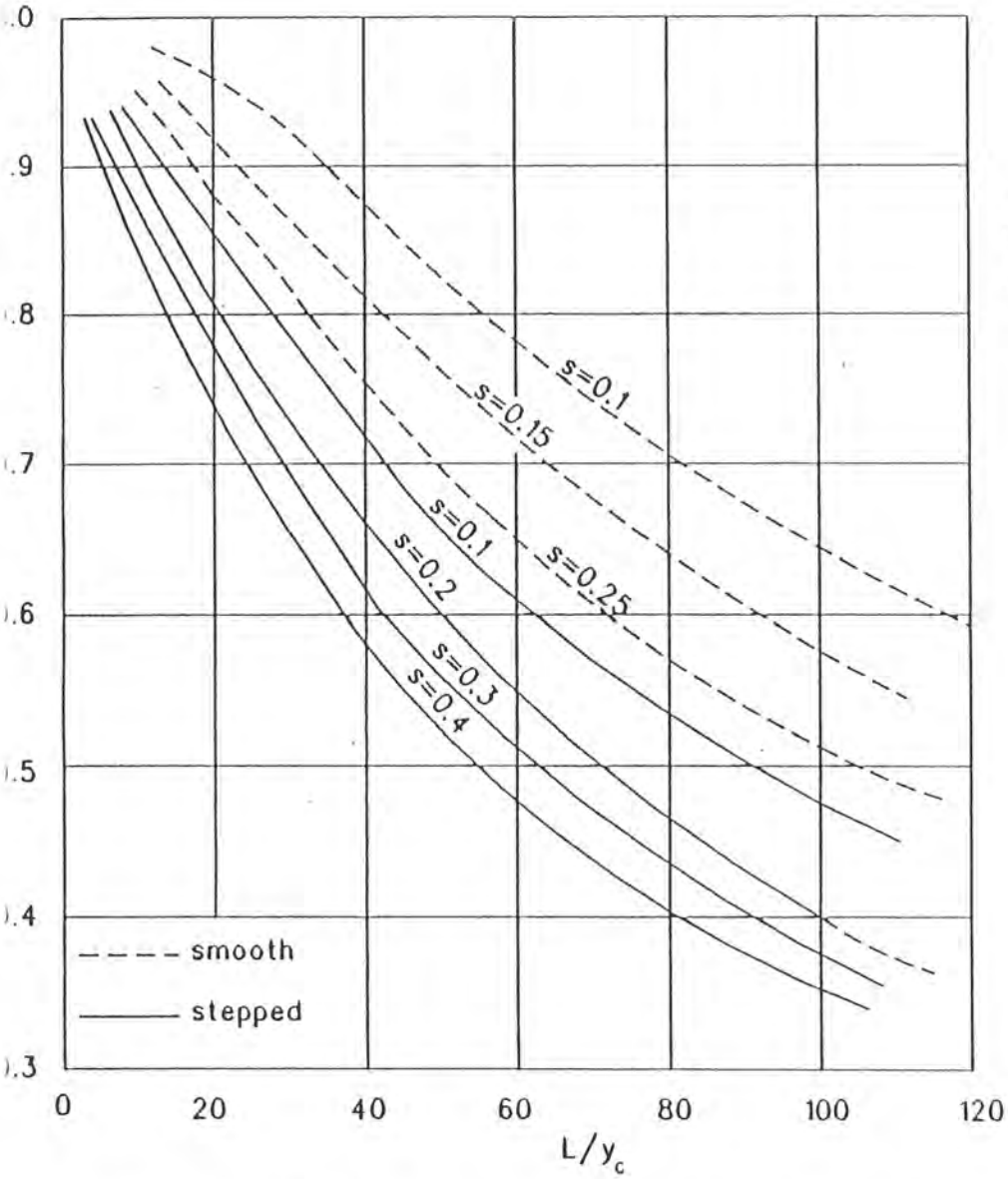


Fig 6.2 Typical block dimensions



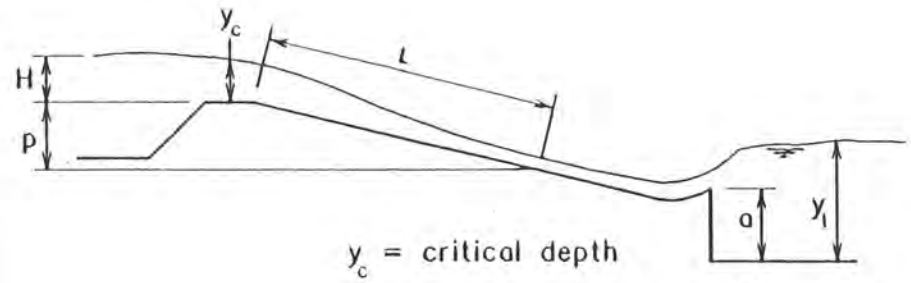


Fig 6.3 Curves For Estimation Of Flow Depth

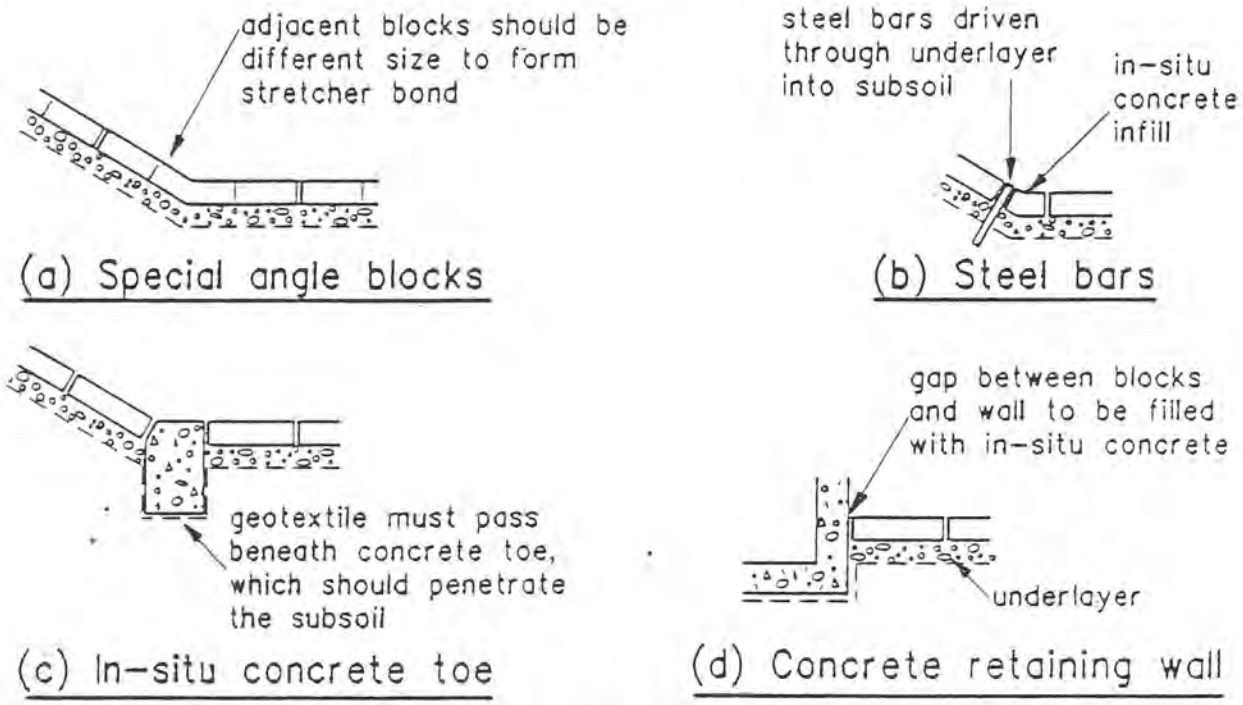


Fig 6.4 Typical Spillway Side Details

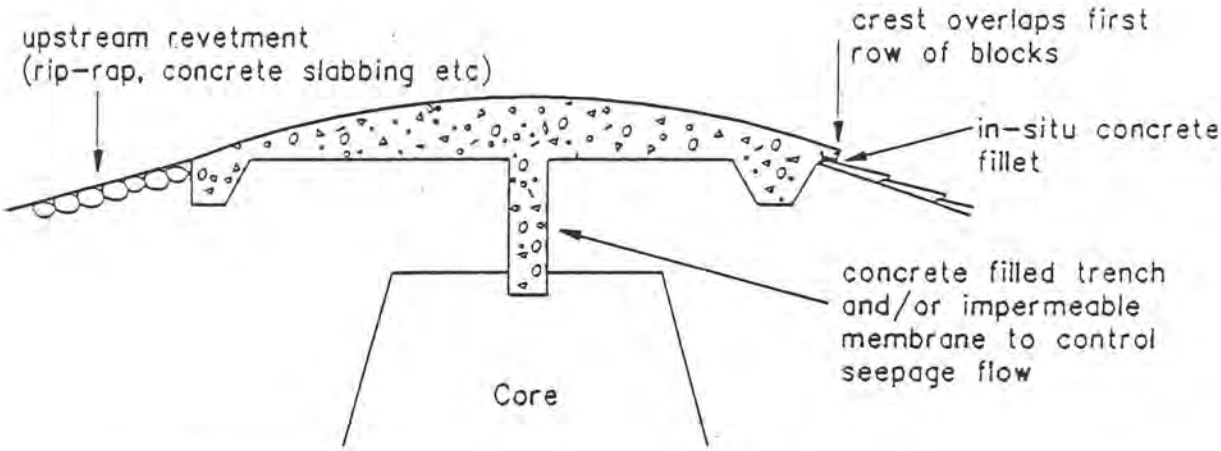


Fig 6.5 Typical Crest Details

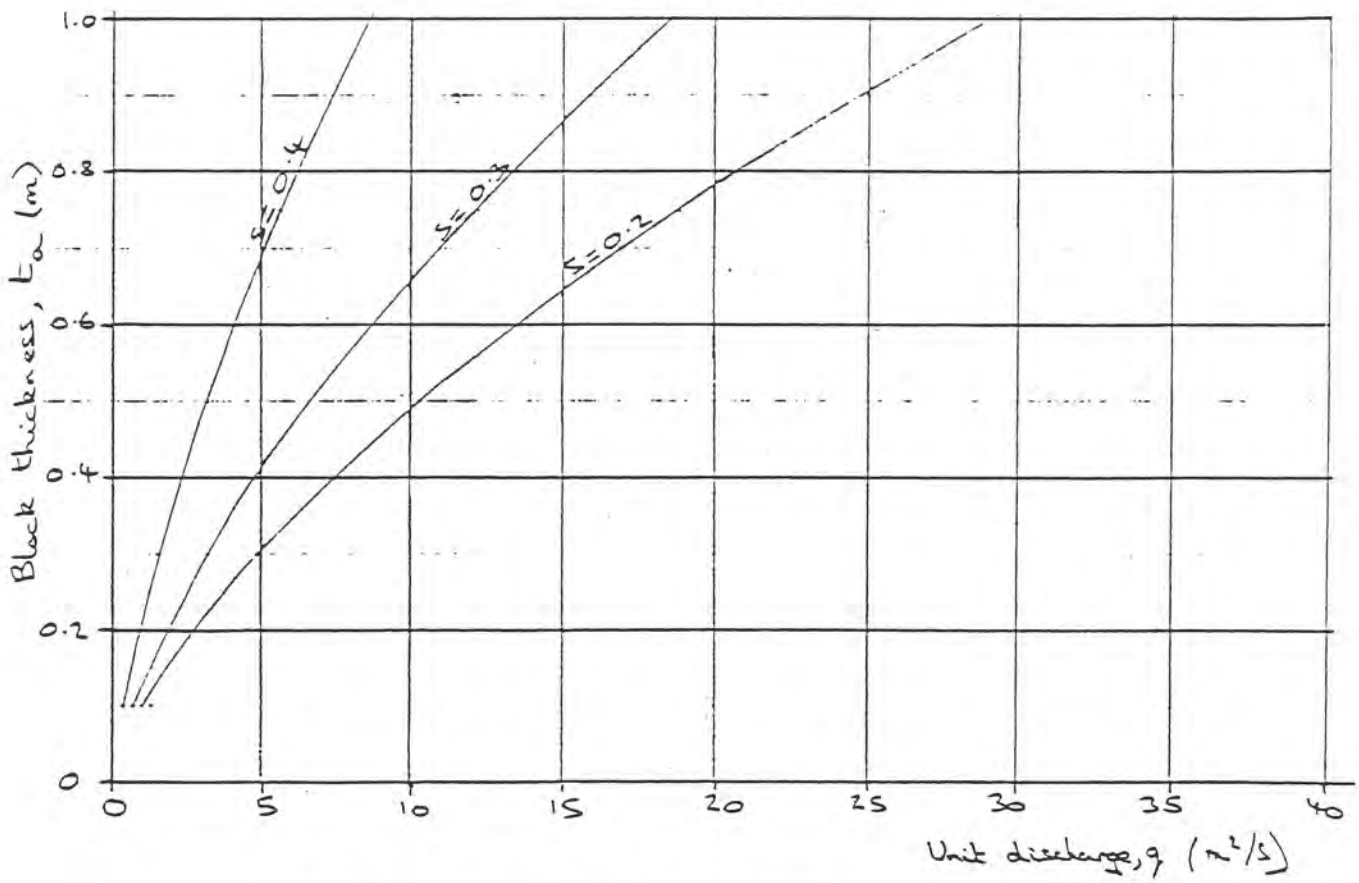
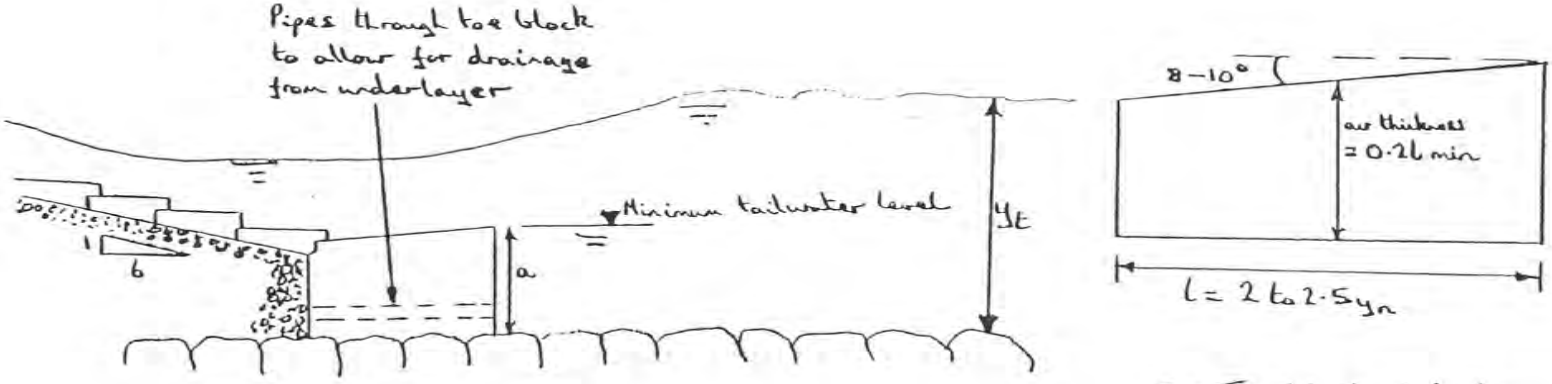
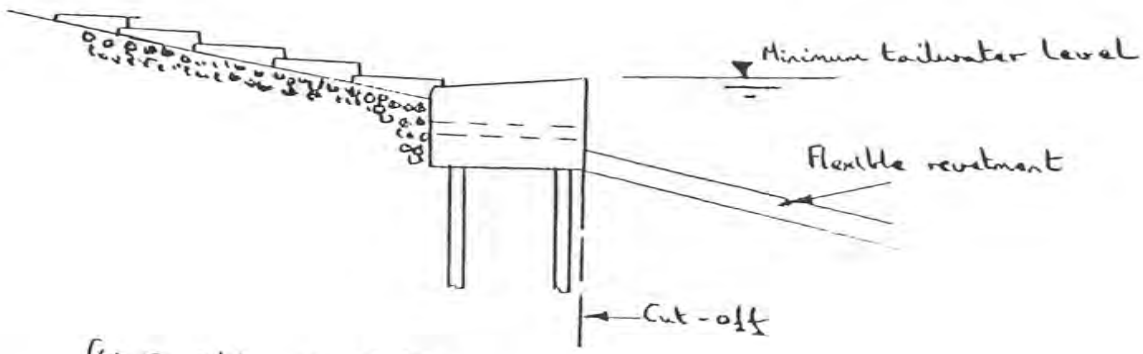


Fig 6.6: Design curves for blocks under a hydraulic jump



(c) Toe block details

(a) Rock foundation



(b) Erodible foundation

Fig 6.7 : Deflector Block at Toe

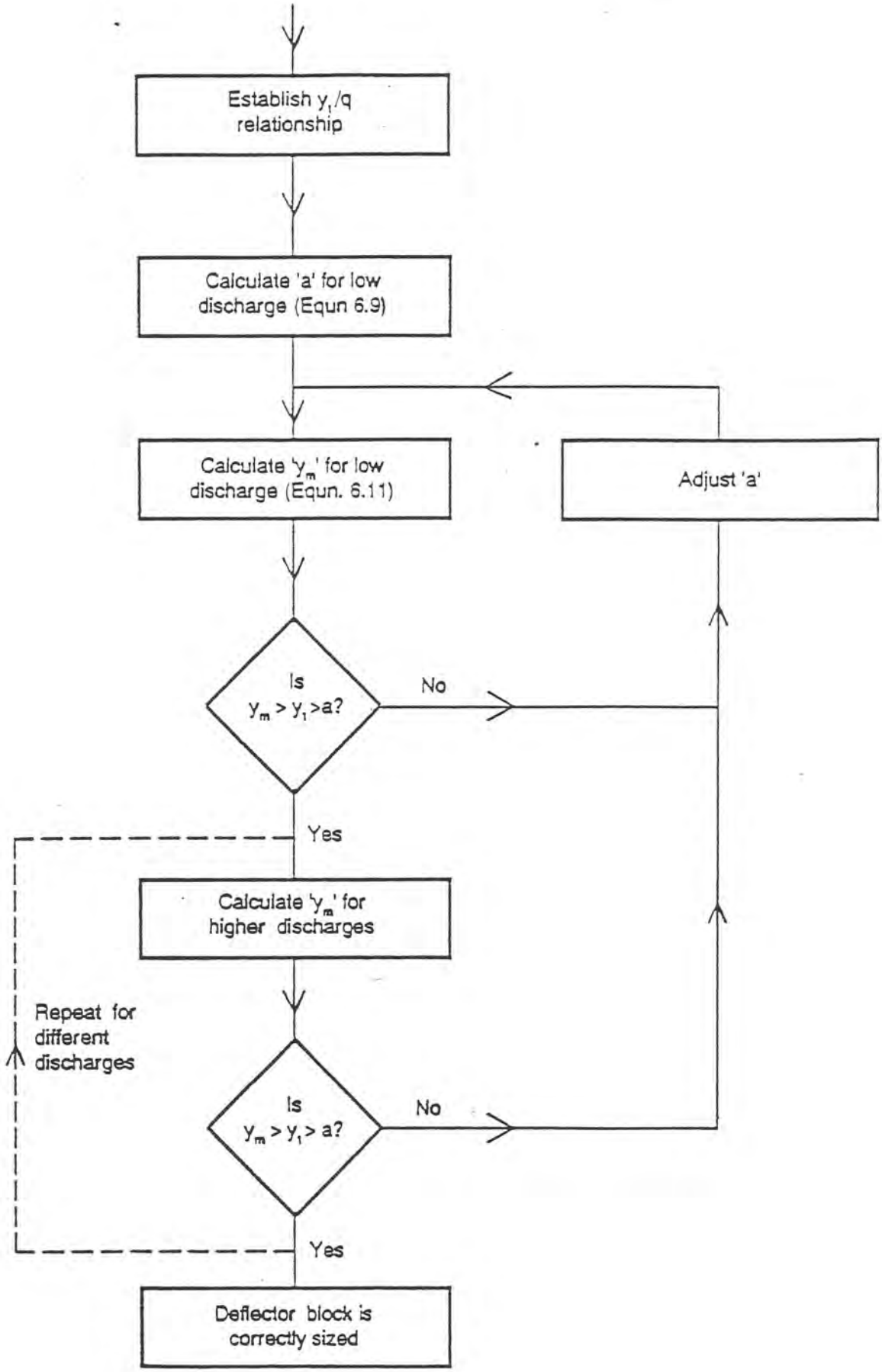


Fig. 6.8: Deflector Block Design Procedure

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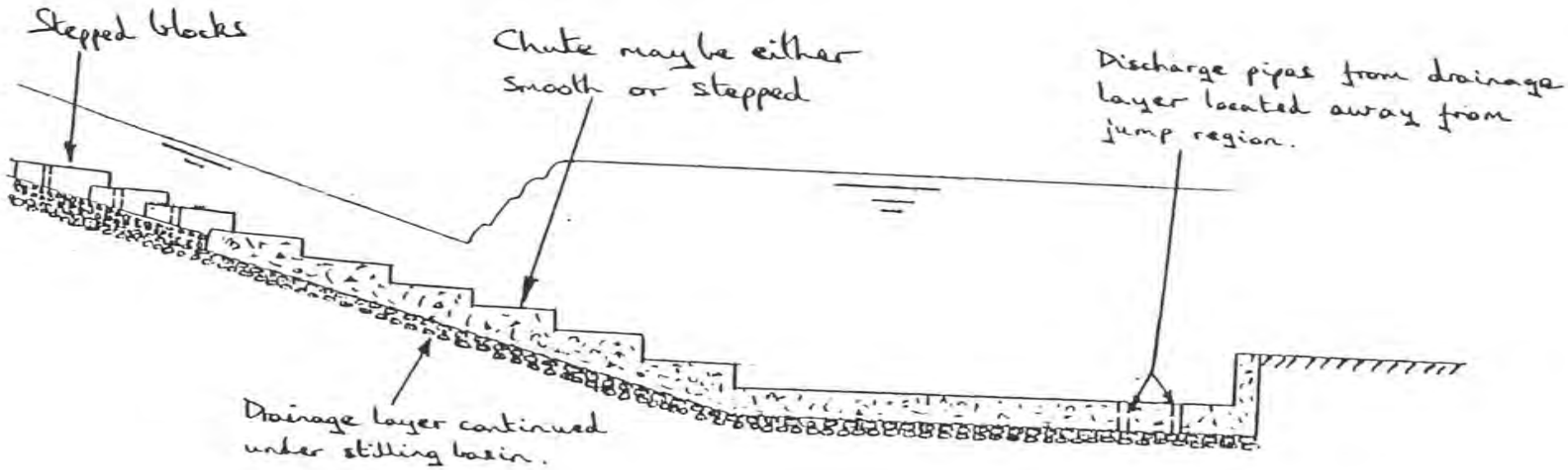

Fig 6.9 In-situ concrete stilling basin

7. SPECIFICATION AND CONSTRUCTION

7.1 Introduction

The design process in Section 6 will result in detailed construction drawings being prepared. In order to ensure that the spillway is constructed as intended by the designer, it will be necessary to provide a written specification to support the construction drawings, and supervise the construction work as it proceeds.

7.2 Specification

The recommended method of specification is:

Specify the required properties or performance of the components,

Indicate general methods of working and acceptance criteria for each stage of construction.

The client and/or designer should decide the extent to which specific products or methods of working are specified. Whatever type of specification is adopted, it is essential that appropriate tests on quality of components and workmanship are included (e.g. sampling and testing of materials supplied to site). Bearing in mind the variation in quality of workmanship which is known to exist in works of this nature, it is recommended that the contractor is required to submit a method statement for approval by the engineer prior to construction of the works.

A check list of points which the designer might include in the specification is given in Table 7.1.

Table 7.1 Specification Guidelines

Section	Item	Coverage
Materials	Precast concrete	Mass; strength; cement; reinforcement; storage; sampling, testing
	In-situ concrete	Mass; strength, aggregates; cement; formwork; reinforcement sampling, testing
	Drainage materials	Type; grading; sampling, testing
	Geotextiles	Trade name, product, grade or minimum/maximum properties; storage, testing
	Other materials	Seeds, fertilisers, imported soils, gabions, cut-off materials
Workmanship	Excavation	Topsoil stripping; bulk excavation; acceptability of formation; trimming of surfaces
	Filling	Methods; compaction; density; moisture content; timing (especially in relation to wet periods)
	Reinstatement	Acceptability of topsoil; application of seed; fertilising; watering; weed control; cutting; remedial work
	Concrete blocks	Contractor's method statement; handling; site damage - criteria for rejection; direction of laying; contact with underlayer; contact between blocks; tolerances - alignment, abrupt changes in profile; shear restraint
	Geotextiles	Positioning - ripples, folds, continuous contact; temporary restraint; cutting; joints and overlays; covering over - timing; trafficking; as-placed records

7.3 Construction

Throughout the design process, the designer must consider the way in which the spillway will be built. It is important that adequate site supervision is provided by the client or his representative to ensure that the works are constructed in accordance with good practice. Particular points that need to be considered are as follows:

1. The method of casting of the stepped blocks. Possible options are:

   (a) dry-casting the blocks in a factory, the concrete being pressed and vibrated. This is likely to be uneconomic unless a large quantity of blocks is required;

   (b) obtaining them wet-cast from a precast concrete manufacturer;

   (c) casting them in moulds on-site.

   Although the wedge-shaped and overlapping type of block would require the most complicated mould it may be more straightforward to cast than other types of block, because the drainage holes would be more difficult to form than the slots in wedge-shaped and overlapping blocks.

2. The method of lifting blocks into position: all but the smallest blocks will have to be lifted by mechanical means, and lifting eyes may need to be cast into the blocks. Alternatively, drainage holes could be adapted for lifting purposes.

3. The method of laying blocks. Construction should commence at the toe and work uphill. Consideration should be given to the gaps between individual blocks: butting the blocks up close to one another could restrict the flexibility of the system and result in voids forming in the underlayer. Leaving too large a gap between blocks could facilitate removal of the underlayer under flow conditions, although the provision of shear restraint would restrain the blocks against sliding.

4. Correct jointing methods should be adopted for geotextiles so that a continuous layer is provided. Adjacent sheets should be either overlapped, sewn or bonded. Where overlaps are in the direction of flow, the upstream geotextile should be laid over the downstream one.

5. Close supervision of construction of granular drainage layers is necessary to check that each layer is of the required thickness and grading and compacted when wet. The layers should be compacted where possible to minimise the possibility of post-construction settlement.

8. INSPECTION AND MAINTENANCE

8.1 Introduction

Once the spillway has been constructed it will be necessary to maintain it to an acceptable standard so that it performs satisfactorily when flow occurs. Consideration should be given at the design stage to inspection and maintenance with a schedule of points to be covered being drawn up and discussed with the owner or other body who will be responsible for maintenance of the spillway.

In the case of spillways for dams which come under the Reservoirs Act 1975, regular inspections will be undertaken by the supervising engineer, the frequency of his visits being recommended by the inspecting engineer. Other stepped-block spillways should be inspected at least once a year by a competent person and after a significant flood event. In the case of spillways which operate infrequently, an inspection should also be carried out immediately after operation.

8.2 Inspection

Particular attention should be paid to the following points when inspecting the spillway:

1. Settlement or other movement (e.g. by sliding) of concrete blocks. A particular advantage of wedge-shaped blocks is that settlement does not result in the leading edge of a block being exposed to the flow. Differential settlement could, however, be a problem between adjacent lateral blocks or in the hydraulic jump zone.

2. Cracking of any concrete blocks. Cracking could lead to a block breaking up and being removed under flow conditions, and removal of one block is likely to lead to the progressive removal of all other blocks.

3. Settlement and cracking of the crest. Particular attention should be paid to the joint between the downstream edge of the crest and the first row of blocks in the spillway to ensure that there is no cracking or movement.

4. Checking that the drainage holes are in good condition: they can easily become blocked with fallen leaves and other rubbish.

5. The underlayer should be probed through the drainage holes to identify any voids between the blocks and underlayer.

6. Any other damage caused by animals, agricultural machinery, vandalism or other means. Vandalism is a widespread problem and could result in deliberate cracking of blocks or blocking of drainage holes.

7. If inspecting after the spillway has operated, the area downstream of the spillway should be inspected closely for any damage due to erosion.

8.3 Maintenance

Because of the interlocking nature of the blocks it will be difficult to replace a broken block with another precast one without temporarily removing several other blocks. It may be easier to replace an individual block using in-situ concrete. A layer of polythene should be placed beneath the block before pouring to prevent the concrete penetrating the underlayer. Drainage holes may be formed with pipes or using a material such as polystyrene which can be burnt out later.

Blocked drainage holes can be cleaned out either using a piece of wire or by burning out the material causing the blockage. Care should be taken not to damage any geotextile beneath the blocks.

APPENDIX 1 - PROTOTYPE INSTALLATIONS

A number of prototype stepped-block spillways have been constructed in the USSR and China and information on these is given in Table A1.1. Further details of some of these installations is given below.

Dneiper Power Station

Wedge-shaped and overlapping stepped blocks were installed in a special test chute constructed in a section of gated spillway at Dneiper Power Station. The test section containing the blocks was 31.6m long and 14.2m wide with a longitudinal slope of s = 1/6.5. Two specially instrumented blocks were included in the test chute. One was equipped with ten pressure sensors located around its perimeter. The other was used to measure the horizontal and vertical forces acting on the block.

The Dneiper test channel was operated under controlled conditions for a total of 10 hours. Flow conditions approaching the blocks were below normal (or uniform) depth, so velocities were higher than those which would occur on a similar channel of constant slope subjected to the same overtopping discharge. Thus, although the blocks successfully withstood a maximum unit discharge of $60m^3/s/m$, the flow velocities were in fact equivalent to those which would occur at $130m^3/s/m$ under uniform-flow conditions. The majority of the blocks were displaced 2cm to 3cm vertically and downslope from their original positions. In two localised areas the deformations reached 0.5m to 0.7m due to the loss of filter material and underlying fill. Granular filter material of the required size (40-100mm) had not been available, so a 750mm thick layer of smaller stone (20-40mm) was used, with protective mesh placed beneath the drainage holes of the blocks to prevent loss of the filter. The mesh was distorted by the flow and the resulting removal of material through the drainage holes was responsible for these two large deformations; despite the considerable movements, the blocks remained in position. Pressures measured around one of the instrumented blocks showed that the maximum fluctuations on the top surface had a standard deviation equivalent to 1.0m head of water; on the underside of the block the standard deviation was only 0.06m. Maximum values of the hydrodynamic forces acting on the second instrumented block were found to be 4.5 tonnes vertical and 2 tonnes horizontal. The success of these full-scale tests showed that suitably-sized stepped blocks can withstand very large flow rates corresponding to overtopping heads of more than 11m; careful design of the drainage holes in relation to the size of the material in the underlayer is necessary. The Dneiper test chute, which is illustrated in Fig. A1.1, is understood to be still in existence. Further details are given in Grinchuk et al (1977).

Dneister Power Station

The stepped blocks used at the Dneister power station were installed, on a trial basis, along part of a 250m long cofferdam. Most of the downstream face of the cofferdam was protected by plane concrete slabs measuring 4.5m long x 10m wide x 0.5m thick. The stepped blocks were installed in a 20m wide test section near the centre of the cofferdam, and their relative economy was demonstrated by their smaller dimensions which were 1m x 1m x 0.25m thick. When the cofferdam was overtopped, scour occurred downstream

of the conventional slabs and partially undermined those in the toe; the slabs did not fail, however, because they were linked by joints which allowed limited rotation. By contrast, downstream of the stepped blocks, bed material was drawn back towards the toe due to the formation of a surface jump. Therefore, at the end of each overtopping season, material that had accumulated downstream of the wedge blocks was excavated and used to fill the scour holes downstream of the conventional slabs. The wedge blocks successfully withstood several floods and two ice-flows which occurred in 1978 and 1979.

Jelyevski Dam

A stepped block spillway was built at Jelyevski dam on one of the abutments where the material was of poor quality clay and sand. Wedge blocks were used at Jelyevski dam in an installation which was designed by a previous student of Professor Pravdivets but without the latter's supervision or involvement. Unfortunately the underlayer was not correctly designed, and consisted of 50mm to 200mm filter material which failed to protect the underlying soil from erosion. As a result, a considerable amount of this material was washed away when the spillway operated, and the channel became approximately horizontal, with the blocks at the upstream end lying well below the spillway crest. Although the system as a whole failed, the blocks themselves remained stable. Professor Pravdivets investigated the reasons for the failure, and published his findings in a technical article, Pravdivets (1982a). The incident confirmed his opinion that stepped block spillways should normally be constructed on the embankment sections of dams and not on the abutments because the fill used in an embankment is carefully selected and placed and therefore has more predictable properties.

Kolyma Dam

Kolyma is a major high-head embankment dam, and the design authority proposed to protect the downstream face with large size rip-rap. As an alternative, Professor Pravdivets suggested the use of stepped blocks measuring 2m x 2m x 0.4m thick, set at a slope of s = 0.4 and able to withstand a unit discharge of $90m^2/s$.

The design authority decided to carry out a comparative test between rip-rap and stepped blocks. A 1:10 scale sectional model of the proposed Kolyma scheme was built in a very large open-air flume in Siberia, downstream of Krasnoyarsk dam. The flume was 5m high, and one half of the model embankment was protected by 2m x 2m rip-rap and the other half by stepped blocks. The rip-rap failed almost immediately, but the stepped blocks performed satisfactorily. Unfortunately, no dividing wall was constructed between the two, so when the rip-rap failed, the stepped blocks were undermined down one side; this prevented further tests being carried out on the blocks. Despite the outcome of the study, stepped blocks were not adopted for the main Kolyma scheme. Details of the 1:10 model are given in a paper by Pravdivets (1981). Stepped blocks were, however, used for a small embankment dam forming part of the overall Kolyma scheme, and this is the installation described in Table A1.1.

State Farm Dams

The spillway channels built at Bolshevik, Klinbeldin, Maslovo, Sosnovski and Zaraysk were for state farms and were constructed using standard rectangular reinforced concrete slabs assembled in stepped fashion. The slopes of the channels are between 1V:5H and 1V:6.5H, and were determined by the characteristics of the embankment material. The design unit discharges are in the range 3.0 to 3.3 $m^3/s/m$, corresponding to overtopping heads of approximately 1.5m. Some of these channels are illustrated in Figs. A1.2 - A1.4.

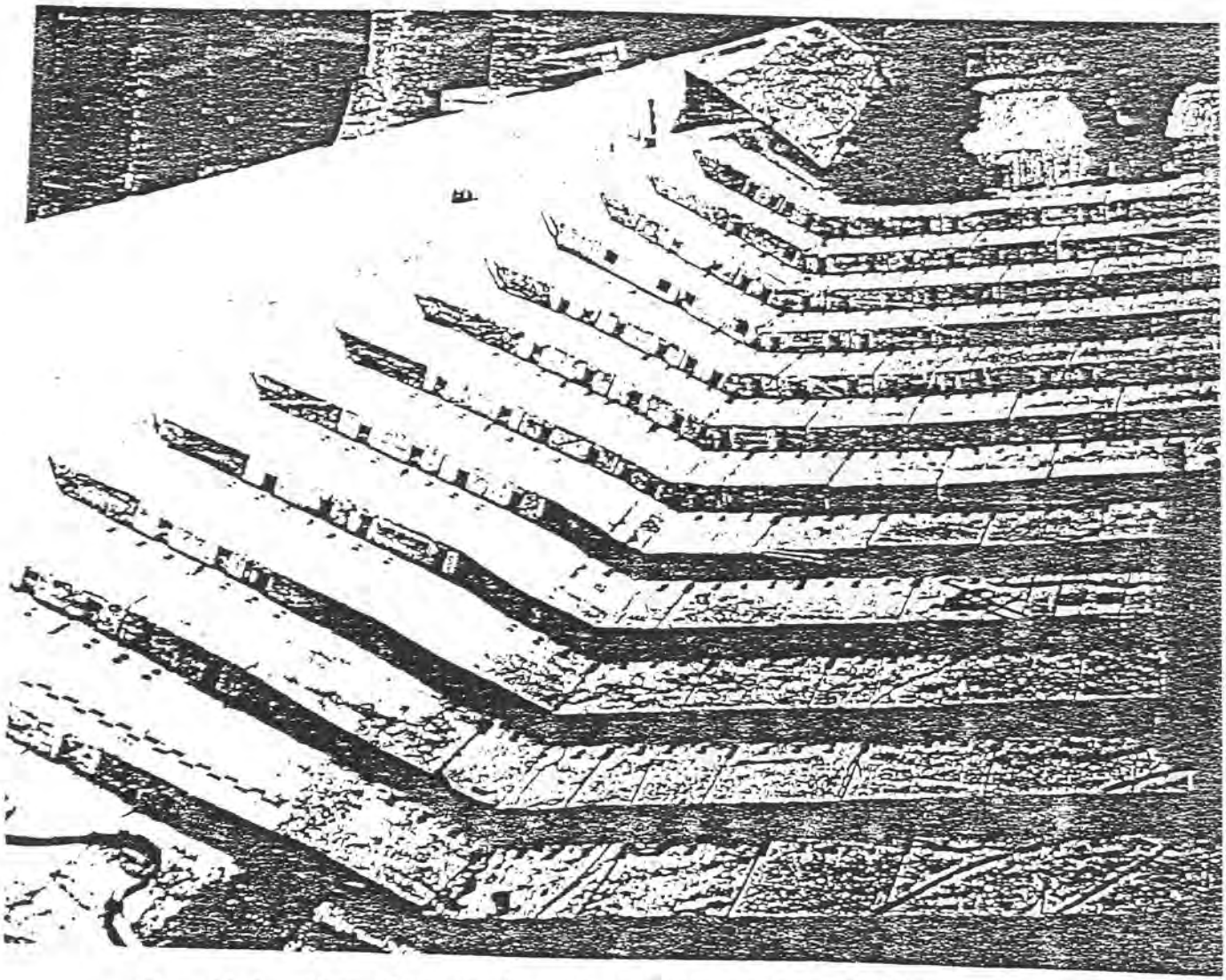

Fig. A1.1: Part of test channel at Dneiper Power Station

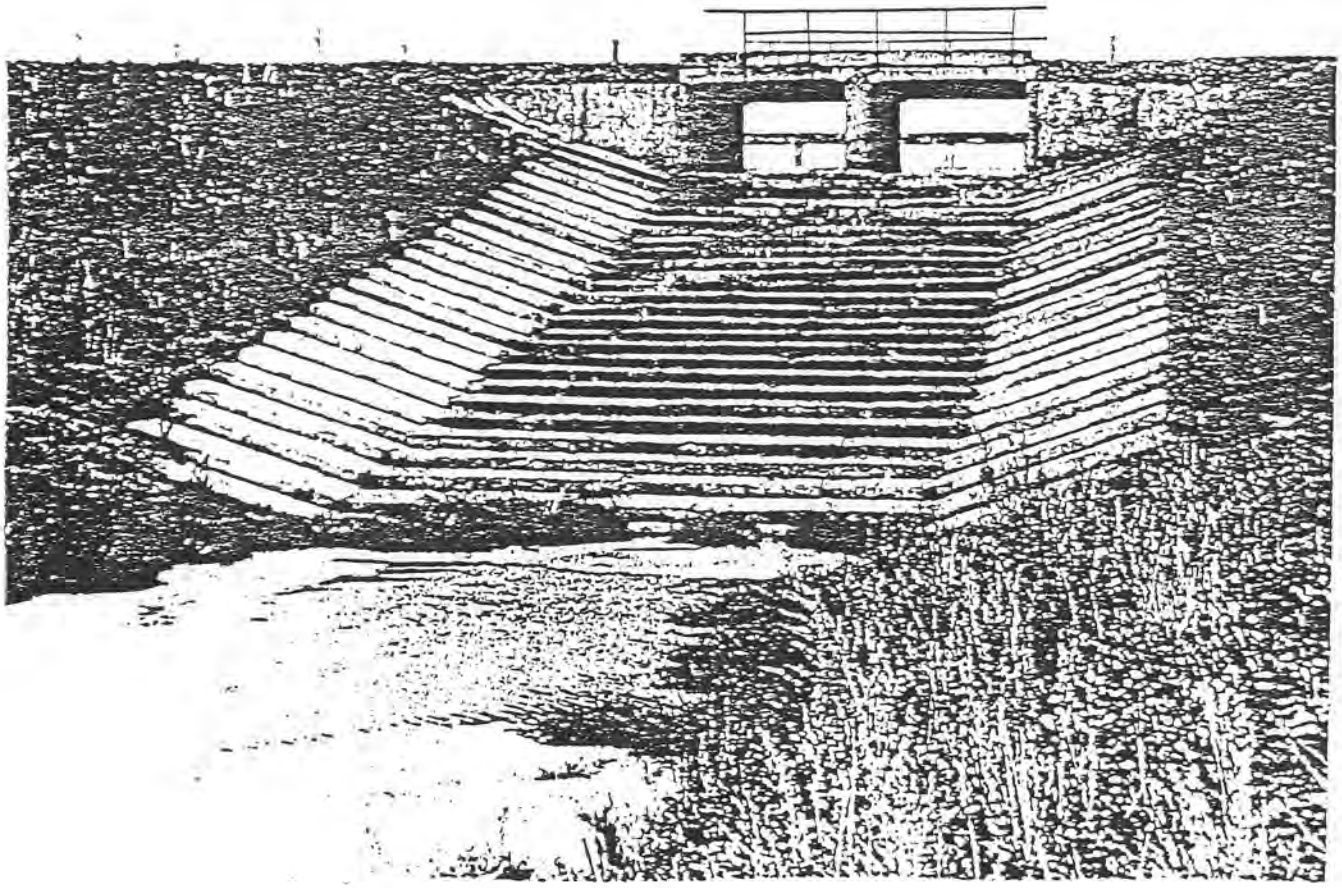

Fig. A1.4: Zaraysk spillway

In the absence of site-specific or calculated values, then:

$$\psi > 5 \times 10^4 k_b$$

using limiting values of $i = 10$, $\Delta H = 0.1m$ and $\psi = 0.002$.

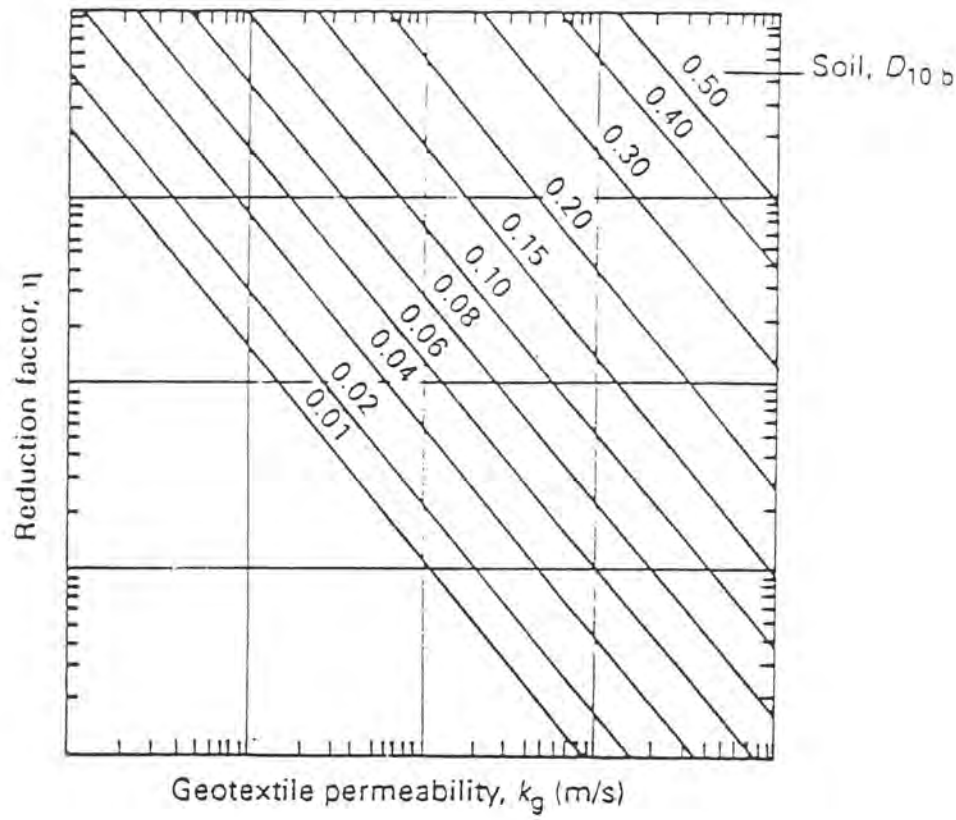

Fig A3.1 : Permeability Reduction Factor, Woven Geotextile Fabrics.

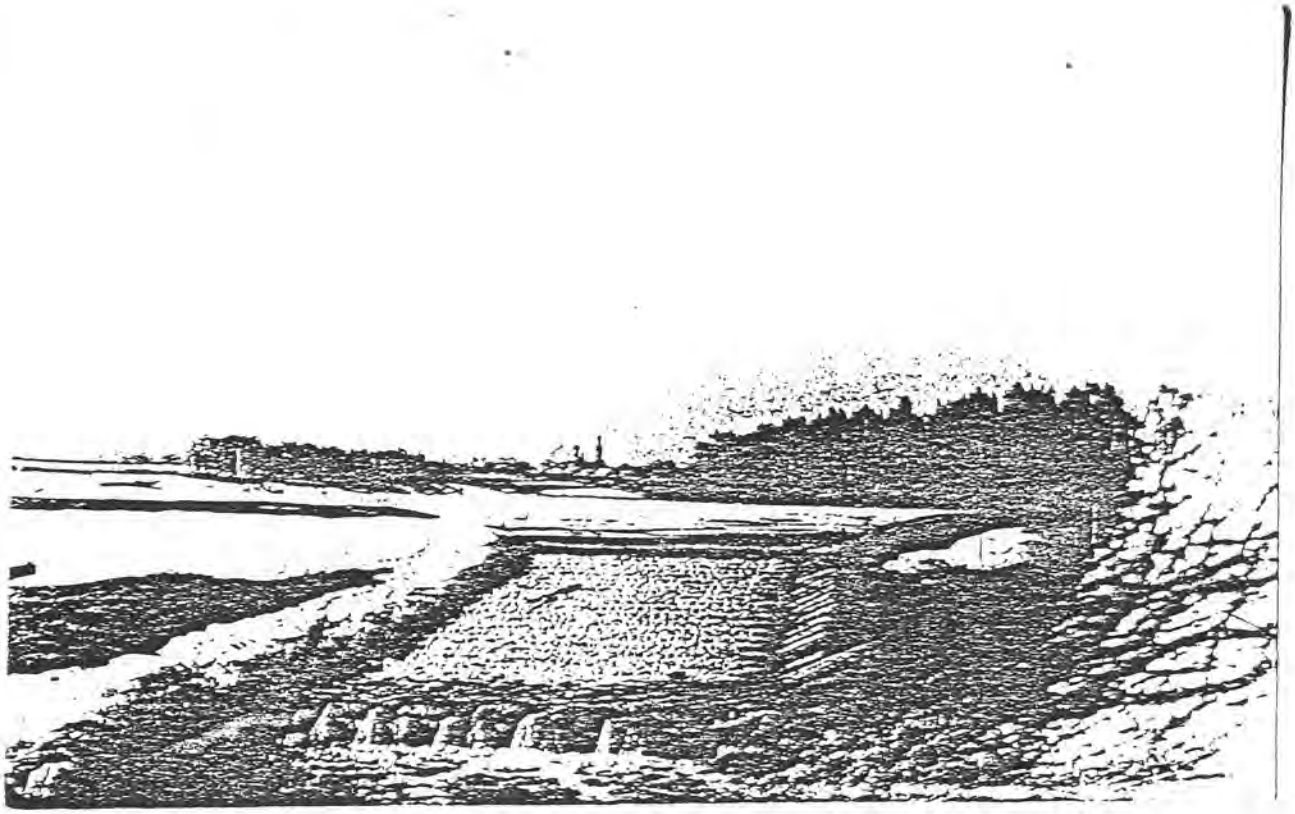

Fig. A1.2: Klinbeldin spillway in operation

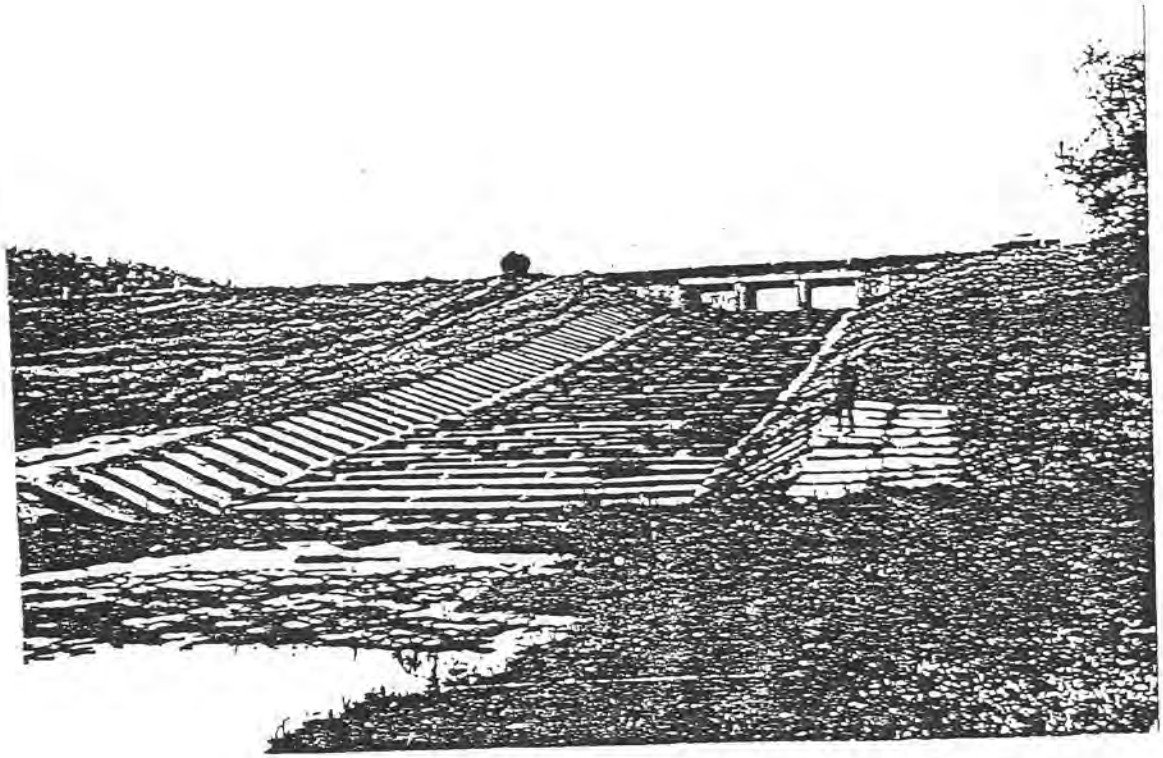

Fig. A1.3: Sosnovsky spillway

TABLE A1.1: Prototype Installations

LOCATION	* BLOCK TYPE	LENGTH L_1, L_0 (m)	STEP HEIGHT Δ_s, Δ_n (m)	WIDTH B (m)	THICKNESS t_s (m)	CHANNEL SLOPE J	CHANNEL WIDTH (m)	HEAD Z_1, Z_0 (m)	q (m^2/s)	V (m/s)	DATE BUILT	COMMENTS	REFERENCES
RUSSIA													
BOLSHEVIK	OS	1.3,1.5		3.0	0.16	0.12-0.2	12	-,11.5	3.3		1980	FARM DAM	KREST'YANINOV & PRAVDIVETS (1986)
KLINBELDIN	OS	1.3,1.5	0.2,-	3.0	0.16	0.159	15	7.5,5.5	3.0	7.5	1976	FARM DAM	PRAVDIVETS (1987), KREST'YANINOV & PRAVDIVETS (1978)
MASLOVO	OS	1.3,1.5	0.2,-	3.0	0.16	0.154	7.5	7.5,5.5	3.0		1981	FARM DAM	KREST'YANINOV & PRAVDIVETS (1978, 1986)
SOSNOVSKI	OS	1.3,1.5		3.0	0.16	0.167	12	13,11	3.3		1978	FARM DAM	PRAVDIVETS ET AL (1980), KREST'YANINOV & PRAVDIVETS (1986)
LUHOVITSY												VERTICAL SIDE WALLS	KREST'YANINOV & PRAVDIVETS (1986)
DNEIPER POWER STATION	WO	2.6,3.0	0.5,-	3.0	0.8	0.154	14.2	37,-	63	23		FULL-SCALE TEST CHANNEL	GRINCHUK ET AL (1977), PRAVDIVETS & SLISSKY (1981)
DNEISTER COFFERDAM	BW	1		1	0.25	0.222	20	7	13	8			PRAVDIVETS (1978a), PRAVDIVETS & SLISSKY (1981)
KOLYMA	OS	2		2	0.25	0.5	6	20,-	5				PRAVDIVETS ET AL (1980), PRAVDIVETS (1987)
TRANSBAIKAL	WO					0.4	115	13,8.5	18				
JELYEVSKI	WO	3		2	0.35	0.125	12		36.5			FAILED	PRAVDIVETS (1982)
CHINA													
JIANGSHE WANAN	BW	2.12	0.52,-	2.12	0.74	0.2							

* KEY: OS - Overlapping slabs BW - Butt-jointed and wedge-shaped WO - Wedge-shaped and overlapping

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APPENDIX 2 - LABORATORY STUDIES

A2.1 Previous studies

A summary of the hydraulic tests that have been undertaken under laboratory conditions is given in Table A2.1. Noori (1985) and El Khasab (1986) investigated the flow resistance using fixed strips arranged in a stepped fashion so as to produce the same external shape as stepped blocks. Laboratory tests on loose blocks have mainly been carried out using blocks measuring about 60mm x 60mm in plan or less. In only one or possibly two studies did failure occur, so the values of q_{max} given in Table A2.1 are the maximum unit discharges to which the blocks were tested. In the case of experiments made by Noori (1985), the failure discharge q_f is the value at which the blocks and the underlayer together started to slide down the sand embankment beneath.

Although many laboratory studies have been carried out in the USSR, detailed experimental data from the tests are not readily available. Many of the studies have been made at the Moscow Institute of Civil Engineering (ICE) under the supervision of Professor Pravdivets (eg. Nguen Dang Shon and M E Lunatsi in Table A2.1). Some of the tests with blocks measuring 20mm x 20mm in plan were carried out in a 180mm wide flume representing a section of a typical embankment dam at a scale of 1:100. The embankment was formed using a coarse sand, and the slope of the downstream face was decreased towards the toe where the blocks were approximately horizontal; the final blocks were angled slightly upwards to deflect the flow away from the bed.

Nguen Dang Shon tested two sizes of block (see Table A2.1) and may have used the 180mm wide flume described above. To provide drainage, each block had one circular hole on the centreline and two semi-circular holes, one down each vertical longitudinal face. The model arrangement reproduced an embankment dam with a 1:2 upstream face and a difference in level between the overflow crest and the tailwater channel of 31.7m prototype; the model scale is believed to have been 1:100. The chute channel formed by the blocks had a slope of $s = 0.35$ over most of its length, but near the toe the slope decreased to zero, with the blocks continued to form a horizontal channel downstream of the toe. Tests were carried out with a hydraulic jump formed over the blocks in the horizontal section of channel. The Type 1 blocks were found to be stable on the sloping section of channel for prototype unit discharges of $q = 100m^2/s$, and in the toe region for values up to $q = 30m^2/s$; the Type 2 blocks were stable in the toe up to $q = 40m^2/s$. (Assuming the scale to be 1:100, these three values would correspond to unit discharges in the model of $0.10m^2/s$, $0.03m^2/s$ and $0.04m^2/s$ respectively).

Tests have been undertaken at Moscow ICE to determine both the hydraulic resistance of stepped blocks on sloping channels and the mean and fluctuating forces exerted on them by the flow. For the latter work, test blocks were mounted on stiff supports with

strain gauges to measure the normal and tangential components of the forces. The sampling frequency was 100 Hz, and spectral analyses of the results showed fairly flat distributions of energy up to the cut-off frequency of 50 Hz but with peaks at frequencies between about 5 Hz to 10 Hz; it is presumed that these were model frequencies.

Further studies of the stability of stepped blocks beneath hydraulic jumps were carried out by another research student, M E Lunatsi. Wedge-shaped and butt-jointed blocks were used to form a horizontal channel in a 460mm wide flume. Conditions at the toe of a spillway chute were simulated by means of an adjustable gate which produced high-velocity horizontal flows over the blocks. Five sizes of block were tested, as detailed in Table A2.1. The area of the drainage holes in each block was varied between 0 and 5.5% of the plan area of the block. Tests were carried out with values of the ratio Δ / y_c in the range 0.2 - 0.5, where y_c is the critical depth of the flow. Froude numbers of the flow upstream of the jump were varied between F = 4.5 and 6.6.

Lunatsi measured pressures on the top faces of the blocks but not shear stresses. It was found that the use of the blocks shortened the length of the hydraulic jump compared with the value for a smooth horizontal channel. Lunatsi suggested that a double wedge-block design would provide greater stability in the jump zone; this would have the same profile as two individual stepped blocks joined together. The double block would be more stable because its greater length would reduce the effect of localised turbulent fluctuations. Professor Pravdivets had earlier patented multiple stepped block designs (e.g. quadruple pattern), but it is understood that model tests have not been carried out.

An earlier study of an alternative type of protection system for erodible embankments was carried out by Professor Pravdivets in about 1969 at the Hydroproject research centre in Volgograd. This system consisted of rectangular blocks linked longitudinally by pairs of flexible link-chains; the blocks therefore forming a type of protective mattress. Flow separation at each block protected the exposed material between it and the downstream block provided the gap was not too large. Tests were carried out on a 1:75 scale model to determine the optimum gap-length/block-length ratio. The system was used at full size with blocks measuring 1m x 1m x 2m x 2m in plan; the gap length was set equal to the block length.

The tests carried out by Simons, Li & Associates (1989) for the US Army Corps of Engineers and the US Bureau of Reclamation formed part of a larger study on methods of protecting the downstream face of overtopped embankments. The experiments were made in a 1.22m wide flume and reproduced the top 1.8m of a soil embankment with an upstream slope of 1V:2H, a downstream slope of either 1V:2H or 1V:3H, and a horizontal crest length of approximately 6.1m. The tests therefore simulated possible prototype conditions for overtopping heads of up to about 1.2m but under controlled laboratory conditions. The size and shape of the blocks were

based on figures suggested by Professor Pravdivets as being suitable for a 1V:3H slope; the thickness was however then increased to allow for a slope of 1V:2H, and the profile was somewhat modified as a result of rounding up the dimensions when converting from millimetres to inches.

A2.2 Studies at Salford University

Laboratory tests for CIRIA were undertaken between 1989 and 1991 at Salford University. The test facility consisted of two tanks, with a vertical difference in level of just over 4.0m and a horizontal displacement of 10.0m, joined together by a 600mm wide, timber acceleration ramp and two by-pass pipelines each containing a valve (see Figures A2.1 and A2.2). Water could be supplied to the upper tank at rates up to 300 l/s ($0.5m^3/s/m$), and the balance of flow between the pipes and ramp could be regulated by adjusting the valves. The angle of the ramp could be altered, but for these tests was set at a fixed slope of 1 vertical to 2.5 horizontal. The lower 2m of the ramp constituted the test area, with perspex sides and a 300mm drop in invert to allow model blocks to be bedded flush with the acceleration ramp on either impermeable or permeable bedding material. The bottom tank was used to still the water before returning it to the laboratory's pumped recirculating system and by closing a gate on the outlet to the tank, tailwater could be created so that a hydraulic jump formed on the test area.

A 4mm thick polyethylene core of the proprietary fin drain Trammel was laid beneath the blocks to simulate an underdrain with a capacity of about 1.1 l/s/m width. The effect of the stepped blocks on the ramp was simulated by fixing triangular strips of timber, with the same dimensions as the blocks under test, onto the drain. This ensured the correct boundary layer, turbulence, air distribution and sub-block flow at the start of the test area.

Three different sizes of block and two different shapes were initially tested as shown in Figure A2.3. From the information provided by Pravdivets, it had been expected that the medium and small-size blocks would fail but in fact they all remained stable at the maximum flow which, for the smallest blocks was equivalent to a prototype flow of 20 $m^3/s/m$ for a 100mm thick block. Fig. A2.4 shows the smaller size of blocks under test conditions.

Pressure tests were undertaken on the largest size of block using a brass test block with 14 pressure tappings that could be connected individually to a single Druck Scanivalve pressure transducer. The pressure distribution on the top of the block showed low pressures in the roller zone in the lee of the step, highly fluctuating positive pressure at the flow re-attachment point and more steady positive pressure on the downstream edge of the step.

Pressures on the underside of the block were measured with different numbers of drainage holes, varying from no holes to a maximum of 14 holes, representing about 5% of the block surface area. The results of these tests are shown on Figure A2.4 and they indicate a considerable advantage in providing 2.5% open area but there is little further benefit in providing holes covering 5% of the block area.

Lifting tests were carried out by attaching a wire to the downstream face of one of the blocks and running this over a pulley with weights being added until the block started to lift. The force needed to lift a block increased with discharge, mainly due to the increased weight of water above the block. It was also established that interblock friction increased the force needed to lift a block by over 30%.

An investigation into the effect of longitudinal joints between blocks showed that a close spacing with minimum gap was desirable. With a large gap water flowed into the joint disrupting the low pressure zone in the lee of the step and hence restricting the suction capability of the holes/slots. Water was allowed free access to the underdrain resulting in high pressures under the block and producing a curve above the 'no holes' line on Fig. A2.4 for a block with 5% holes. In addition a large longitudinal joint exposes the underdrain material leaving it open to be damaged or washed away, and loses the inter-block restraint described above.

Sliding tests were carried out on the medium-size blocks and with no toe restraint the whole panel of blocks slid across the plastic fin drain at very low flows. The coefficient of dynamic friction between the model blocks and fin drain were measured as 0.78. However, if the flow was raised quickly, the overlapping block shape became stable and overlapping blocks were run in the rig at the maximum flow with a row missing and only slid when the flow was turned off. This did not occur with non-overlapping blocks.

A row of special blocks was made to investigate the step height to length ratio. The bases of the blocks were cast in concrete and then timber sheets of various thicknesses were attached on top to form the tread of the step. These blocks were placed in a row immediately upstream of the pressure-test block, and pressure readings on the upper surface tappings were averaged to give a net pressure on the top surface. The results of these tests are shown in Figure A2.6. These tests confirm Pravdivets' suggested step height to block length ratio of between 1:4 to 1:6. With a small step, the roller zone is too small to be effective. With a large step the blocks became unstable as the flow trajectory ceases to land on the next block downstream and hence the roller zone covers the whole of the block top surface. At the optimum step height to block length ratio of 1:5 the roller zone is at its optimum with the maximum holding down force.

Tests were also carried out on the blocks under hydraulic jump conditions, and two failure modes were observed: individual blocks vibrating normal to the embankment surface and groups of blocks waving up and down as a panel. In both cases an individual block was eventually removed. Once a block was missing two further failure modes were observed: blocks vibrating parallel to the embankment and blocks vibrating transverse to the embankment working their way out of the interlock of stretcher bond into the void created by the missing block, and eventually being removed. These failures occurred at low flows and confirmed Pravdivets' recommendation that a hydraulic jump should not be allowed to form on blocks designed for free flow conditions.

A toe block of the type recommended by Pravdivets was installed at the end of a panel of blocks. The toe block is designed to form a surface jump in the channel downstream of the spillway with the stepped blocks always being subject to high velocity turbulent flow. The behaviour of the toe block under different tailwater conditions is shown in Figure A2.7. The tests confirmed that the design of the toe block was satisfactory provided that tailwater levels did not rise to a level that results in a hydraulic jump forming on the stepped blocks.

Salford University also undertook a model test of a proposed stepped-block spillway to be constructed in Oman which was designed by a British consultant. The blocks for this scheme were wedge shaped and overlapping with the addition of a side-to-side interlock which added to the stability. Drainage through these blocks was by means of slots between blocks and due to high seepage pressures and the difficulty of providing significant drainage area whilst maintaining the integrity of the concrete, the pressure under the block did not draw down with increasing flow rate as shown on Figure A2.4. Subsequent tests in the CIRIA test facility with blocks with 2.8% open area in seven slots have shown that slots can be effective. They have an advantage that less water enters the underdrain at low flow eliminating the peak at 50 l/s on Fig. A2.4 but it is difficult to provide a significant slot area and hence a block with slots could be expected to behave no better than the '6 holes' line on Fig. A2.4.

TABLE A2.1: Data on Laboratory Tests

REFERENCE	BLOCK* TYPE	L (mm)	L_o (mm)	B (mm)	Δ_s (mm)	t_s (mm)	Δ_n (mm)	t_n (mm)	J	q_{max} (m^3/s)	q_f (m^3/s)	MODEL "SCALE"	COMMENTS
EL KHASHAB (1986,87)	BW	40	40	-	4				0.1				Fixed strips
	BW	65	65	-	13				0.2				fixed strips
	BW	24.2	26.0	24	-		3	7.5	0.211	0.10	0.107		
	BW	24.2	26.0	24	-		3	7.5	0.083	0.18	-		
JIANG (1985)	BW	63	63	63	12	12			0.2	0.18		1:20	Aluminium blocks
	BW	53	63	53	13	18.5			0.25	0.241		1:40	sg = 2.7
NOORI (1985)	BW	40	40	-	4				0.1				Fixed strips
	OS	65	85	62.5	13	13			0.2				fixed blocks
	BW	63.5	63.5	63.5	13	12.5			0.2		0.156		aluminium blocks sg = 2.7
BAKER (1989,1990,1991)	BW	52.2	52.2	50			15	17.5	0.4				
	WO	50.7	60	50			8+0.5 gap	15.8	0.4	)0.53			Sand/cement blocks
	WO	25	30	25			4+0.3 gap	7.9	0.4				sg = 2.05
	WO	149	180	150			28+1.8 gap	51.1	0.4				
	WO	200	240	200			40	60	0.286	0.48		1:5	Side to side interlock
SIMONS, LI & ASSOC (1989)	WO	164	205	203			44.5+16.5 gap	122	0.33 0.5	2.125			
<u>RUSSIAN TESTS</u>													
MOSCOW ICE		20 40		20 30				3 to 6 5.1	0.17 to 0.33	0.10		1:30,50 100,120	sg = 2.4
ALL-UNION RESEARCH INE									0.25			1:6,10	
NGUEN DANG SHON	WO WO	 28	25	20 22	2.5 4	7 8			0.37 0.37	0.10 0.10	0.03 0.04	1:100	Blocks did not fall on slope, values of qf refer to failure under jump on horizontal toe
LUKATSI	BJ	89.2					17.5	21.2	0				Tests with jump forced on horizontal channel
	BJ	78.5					15.4	22.3	0				F = 4.5 - 6.6
	BJ	62.4					12.2	23.9	0				
	BJ	46.4					9.1	25.4	0				Δ/y_c = 0.2 - 0.5
PRAVDIVETS (1981)												1:10	Tests in large flume downstream of Krasnoyarsk Dam

* <u>KEY:</u> OS - Overlapping slabs
BW - Butt-jointed and wedge-shaped

APPENDIX 3 - DESIGN OF GEOTEXTILE FILTERS

The filter rules given below are based on Hemphill and Bramley and cover the main principles, but do not provide a comprehensive review. The designer should also consult other references (e.g. PIANC, 1987a; van Zanten, 1986) and manufacturers' source data when designing and specifying a geotextile filter. Filter properties are generally specified in terms of fabric type, pore size and permeability.

Under steady flow conditions, for soil retention:

$$O_{90} < \lambda D_{90b}$$

where values of range from 1.0 to 2.0, depending on soil density and uniformity. Soils with uniformity coefficients greater than 5 are considered to have better self-filtering properties, PIANC (1987a) suggest a lower limit of 50 μm (0.05mm) on minimum opening size.

Further criteria for O_{90} are given by Ingold (1984) and Heerten and Wittman (1984), who also recommend that the maximum opening size should not exceed about 0.3 to 0.5mm.

For geotextile permeability, the design approach must take into account the reduction in permeability of the as-manufactured fabric due to clogging and/or blocking of the fabric pores in service. Geotextile permeability can either be specified so as to ensure that it remains equal to or greater than the permeability of the subsoil, k_b, or can be designed to ensure that the head loss across the geotextile is limited to an acceptable value.

Heerten and Wittman (1984) give an empirical method for assessing permeability reduction factor η, due to clogging/blocking which is applied to the as-manufactured permeability, k_g.

1. For non-woven fabrics (needle punched or thicker than 2mm);

$$\eta = 0.02$$

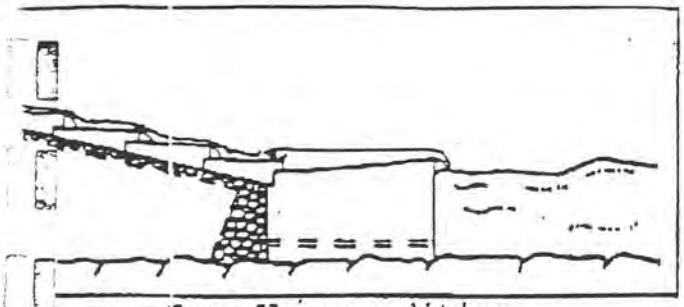
2. For woven fabrics, the reduction factor is itself dependent on as-manufactured permeability and soil particle size, D_{10b}, as shown in Figure A3.1. This criteria may also be applied to thin non-woven fabrics.

After the reduction factor has been estimated, the permeability of the geotextile in service can be compared with that of the base soil.

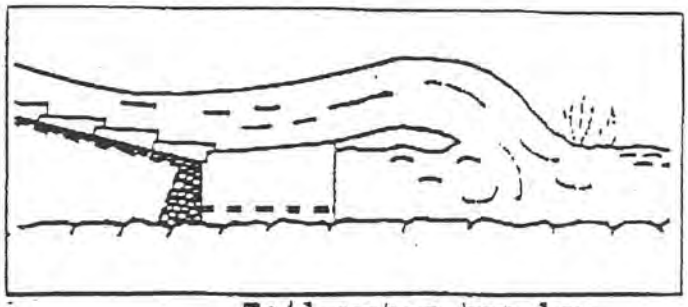
PIANC (1987a) provide a method for assessing geotextile permittivity. Assuming the rate of flow through the soil next to the geotextile is equal to the rate of flow through the geotextile:

$$\eta \psi \Delta H = k_b i$$

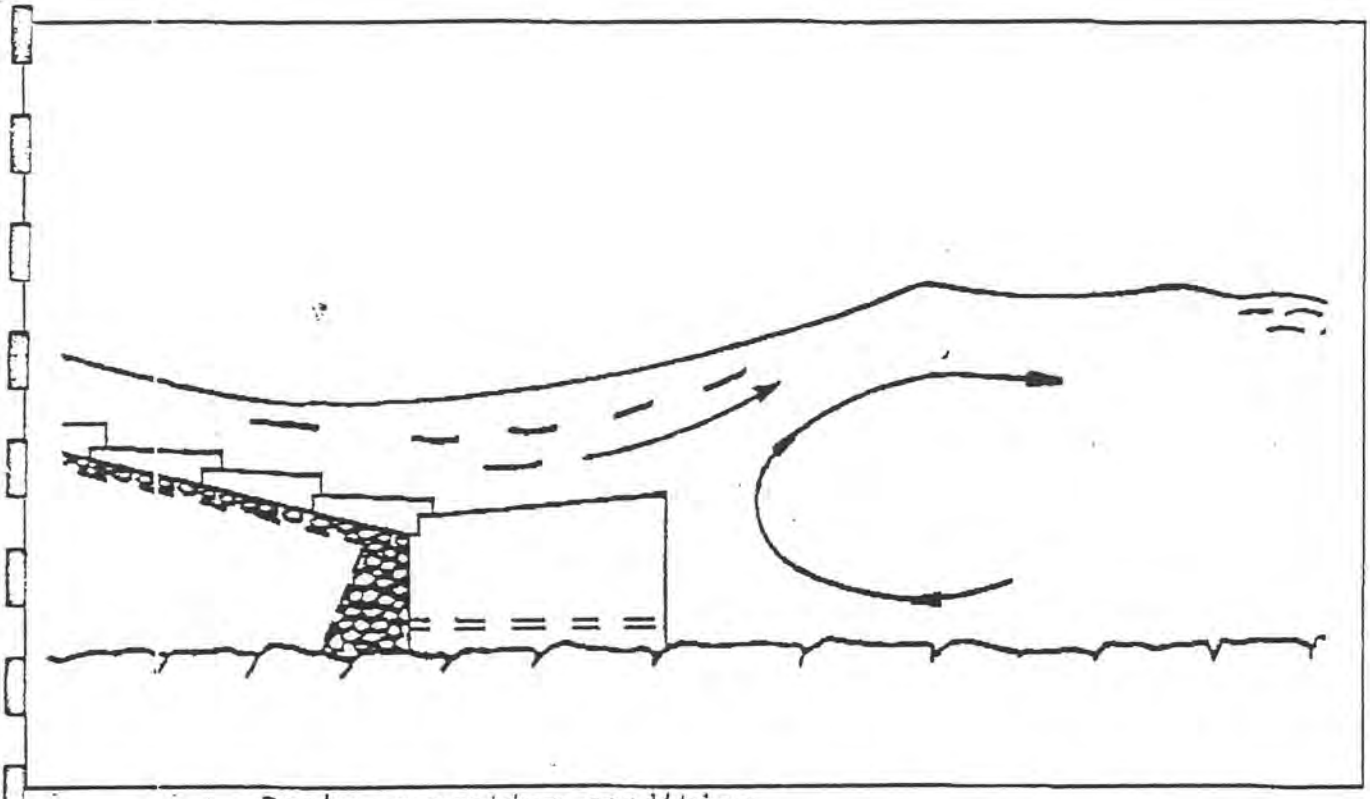
where ΔH is the head loss through the geotextile, and i is the hydraulic gradient in the soil.



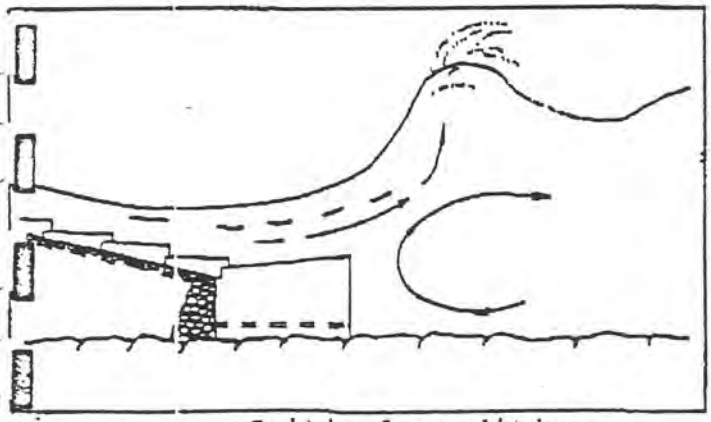
Low flow condition



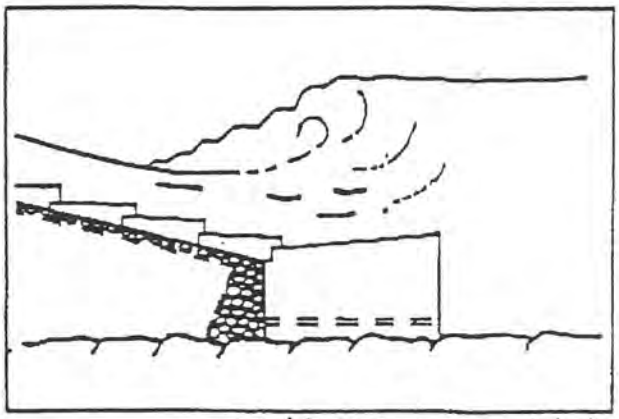
Tail water too low



Design operating condition



Critical condition



Tail water too high

FIGURE A2.7 : Effect of toe block.

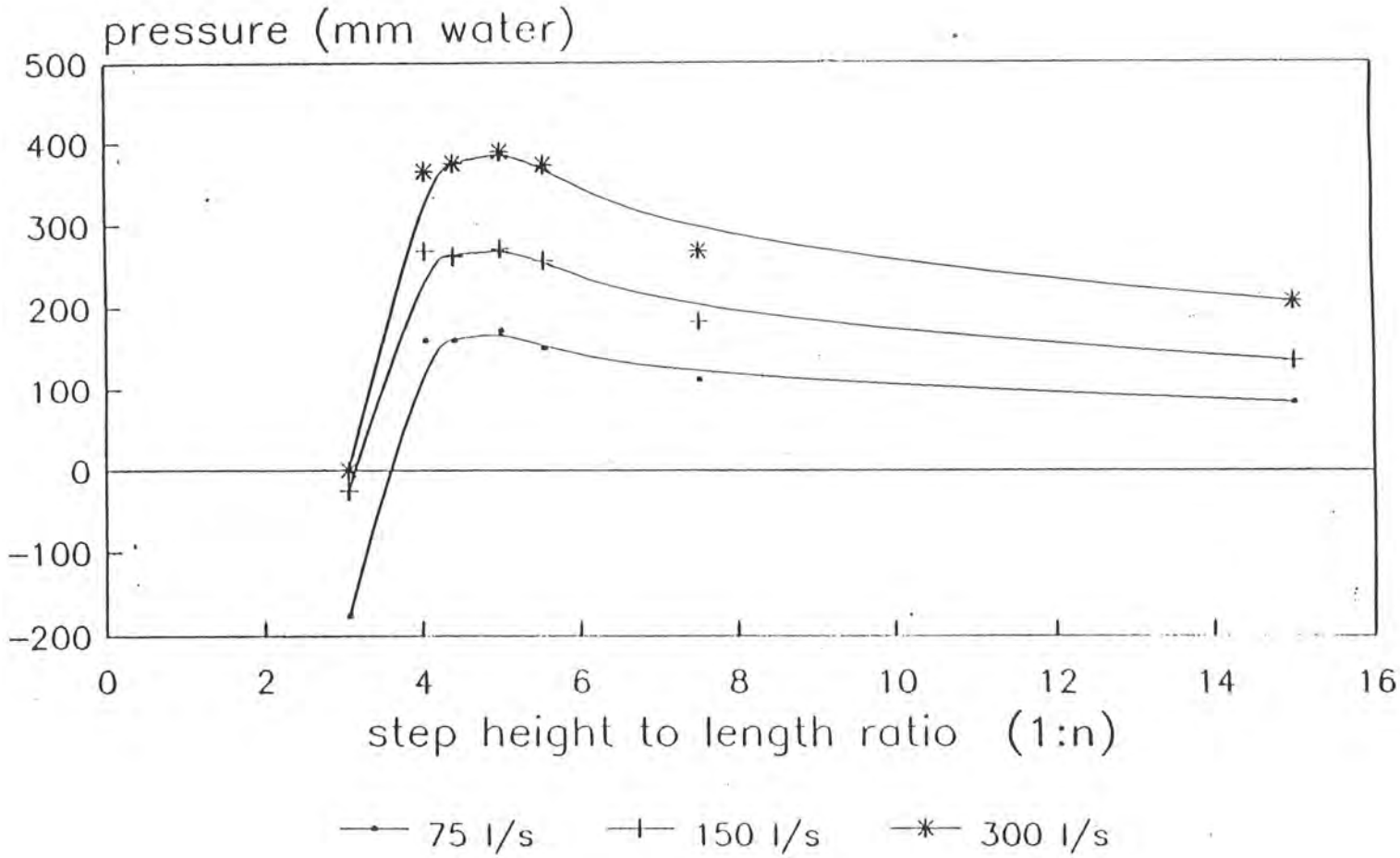


Figure A2.6 Step Height to Length Ratio

Fig. A2.5: Small blocks under test

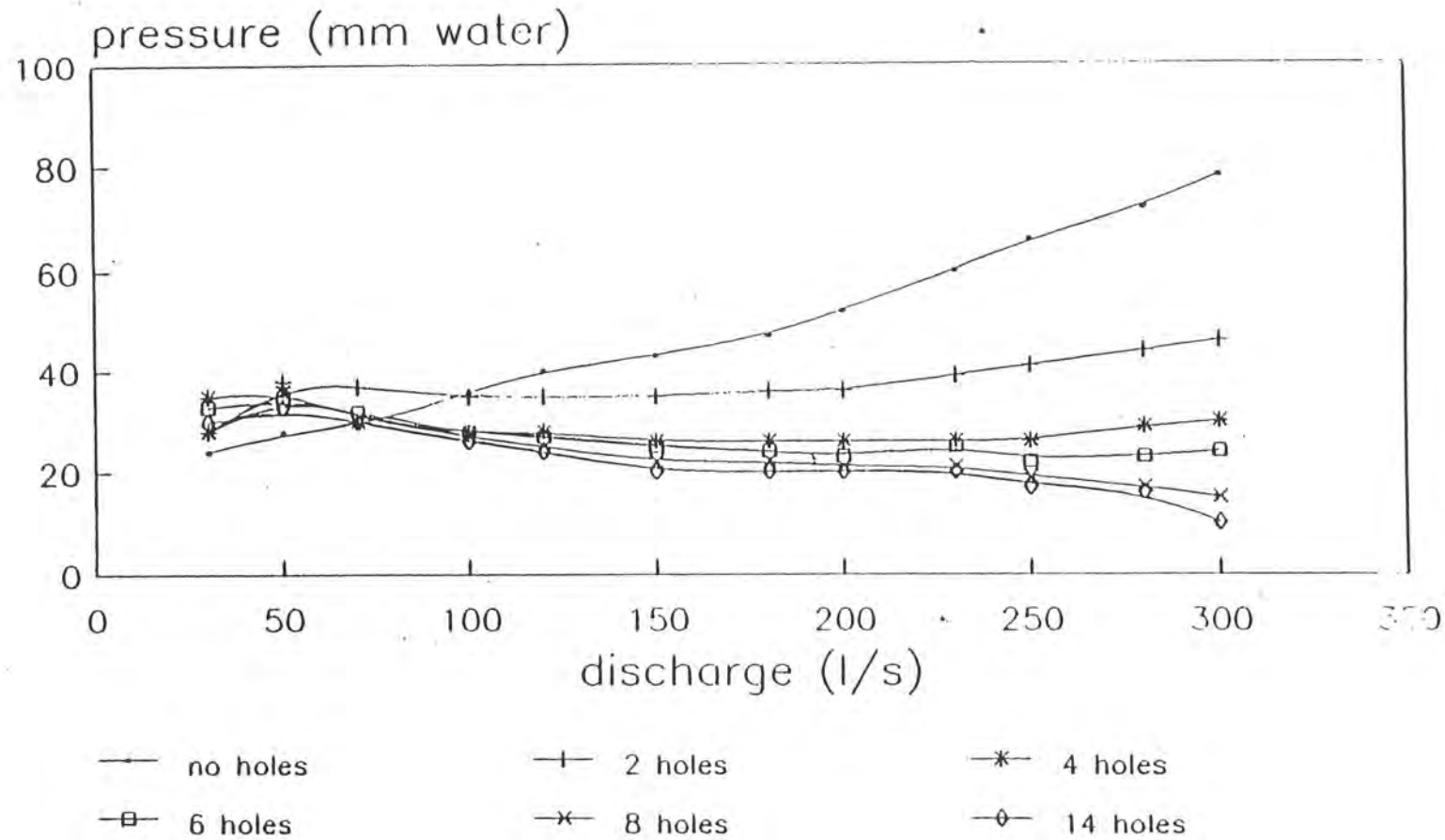


Figure A2.4 Pressure under the block

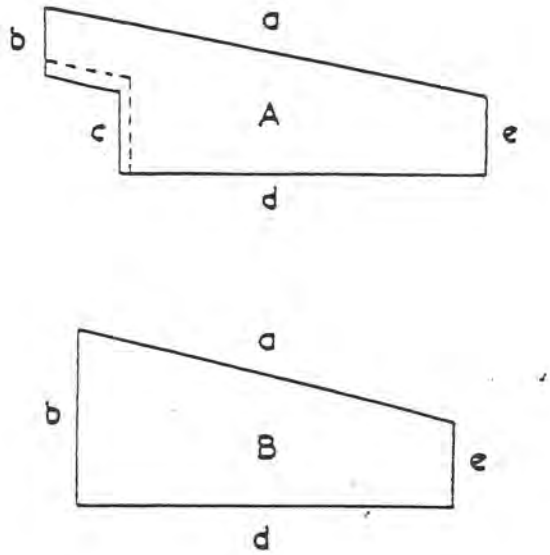


Figure A2.3 Shapes of test block

Size	BLOCK A Small	BLOCK A Medium	BLOCK A Large	BLOCK B Medium
a	30	60	180	50
b	4.5	9	31	23
c	5.5	11	29	-
d	24	48	146	49
e	5	10	30	11
width	25	50	150	50
t_{av}	8.5	17	50	17

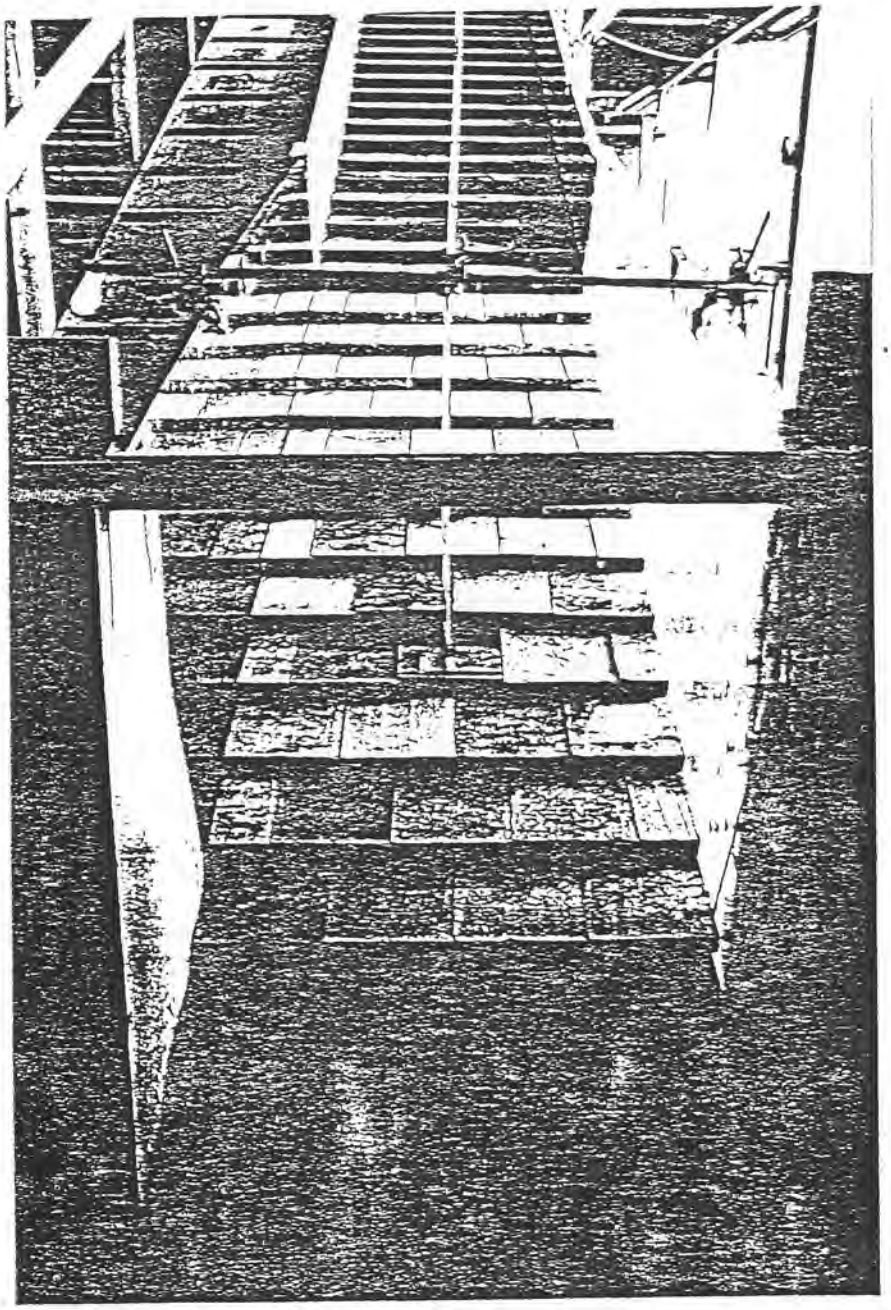


Fig. A2.2: Photograph of test facility

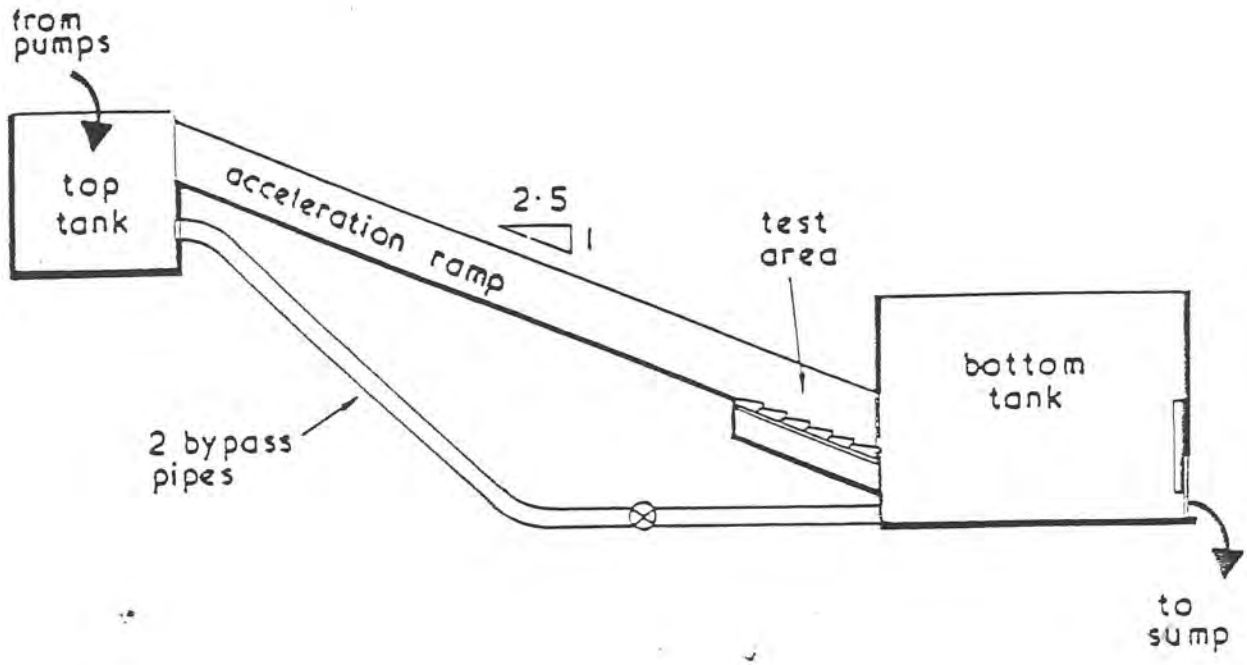


Figure A2.1 Laboratory test facility

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